

EVALUATION OF TIME-DEPENDENT CAMBER AND DEFLECTIONS IN BRIDGE GIRDERS

by

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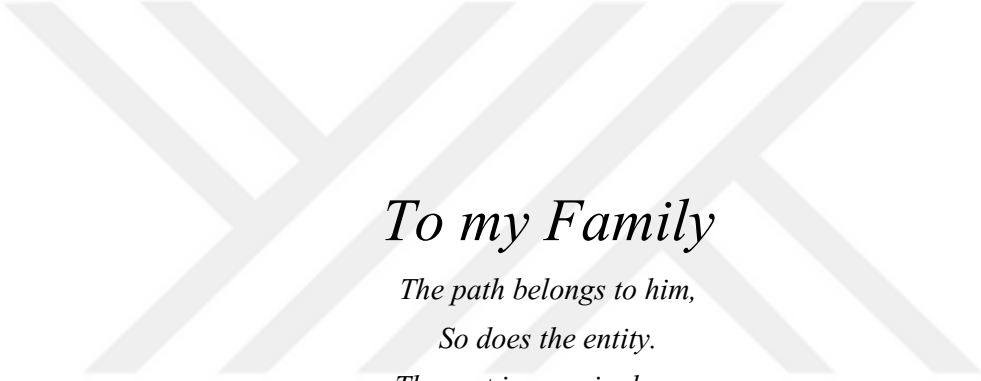


04-27-2021

Advisor

Date

DEDICATION



To my Family

The path belongs to him,

So does the entity.

The rest is meaningless...

N.F.K.

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TABLE OF CONTENTS

DEDICATION.....	ii
ACKNOWLEDGEMENTS	iii
LIST OF TABLES	xiv
LIST OF FIGURES	xvi
CHAPTER 1 : Introduction.....	1
1.1. Background.....	1
1.2. Research Objectives.....	4
1.3. Scope of Research.....	4
1.4. Organization of Thesis.....	5
CHAPTER 2 : Literature Review	7
2.1. Introduction.....	7
2.2. Summary of Previous Studies.....	8
2.3. Methods to Measure Deflection on Members.....	9
2.3.1. Tape Measure Method	10
2.3.2. Stretched-wire Method.....	11
2.3.3. Surveying Method.....	13
2.3.4. Photogrammetry.....	15
2.4. Methods to Predict Camber at Release (Initial Camber)	16
2.4.1. Moment-Area Theorems with Elastic-Beam Theory.....	16
2.4.2. Direct Integration Method.....	24
2.4.3. Conjugate-Beam Method	26
2.4.4. Virtual Work Method.....	28
2.4.5. Tabulated Equations.....	30
2.5. Methods to Predict Long-term Deflections	32

2.5.1. Multiplier Method	33
2.5.2. Incremental Time Step Method (Nilson 1987)	34
2.5.3. Axial Strain and Curvature Method (Ghali and Favre 1986).....	39
2.5.4. Approximate Time Step Method (Branson and Ozell 1961)	40
2.5.5. Prestress Loss Method (Tadros et al. 1985).....	41
2.5.6. CEB-FIB Model Code (CEB-FIP 1990).....	41
2.5.7. Section Curvature Method (ACI 435R-95).....	41
2.6. Factors that Affect Camber at Release (Initial Camber) and Long-term Camber, Deflection and Rebound	42
2.7. Prestressed Concrete Beam Questionnaire	43
CHAPTER 3 : Prediction Methodology	49
3.1. Introduction.....	49
3.2. Time-Dependent Cross-sectional Curvature Method	49
3.3. Primary Assumptions and Sign Convention.....	52
3.4. Prediction Methodology for Camber at Release.....	55
3.4.1. Approaches No. 1 to 4	56
3.4.2. Approaches No. 5 and No. 6.....	60
3.5. Prediction Methodology for Long-term Deflection.....	66
3.5.1. Approach No. 1	66
3.5.2. Approach No. 2	77
3.5.3. Approach No. 3	89
3.5.4. Approach No. 4	95
3.5.5. Approach No. 5	97
3.5.6. Approach No. 6	101
3.6. Modulus of Elasticity Prediction	126
3.6.1. Models for Modulus of Elasticity Prediction at Release and 28 Days.....	126

3.6.1.1. AASHTO LRFD 2020	126
3.6.1.2. ACI 318-19	127
3.6.1.3. ACI 363R-92.....	127
3.6.1.4. fib MC 2010.....	128
3.6.2. Prediction of Modulus of Elasticity as a Function of Time	129
3.6.2.1. ACI 209R-92.....	129
3.6.2.2. fib MC 2010.....	132
3.7. Creep and Shrinkage Models.....	133
3.8. Strand Relaxation Loss	141
3.9. Elastic Shortening Loss	144
3.10. Temperature Gradients	146
3.11. Limitations of the Proposed Prediction Methodology	149
3.12. Time-Dependent Multiplier Method.....	150
CHAPTER 4 : Beam Camber and Displacement Data Sets	152
4.1. Introduction.....	152
4.2. Camber and Displacement Data Sets Provided by MDOT.....	155
4.3. Camber and Displacement Data Sets Collected from Precast Fabricators	159
4.4. Camber and Displacement Data Sets Obtained from the Literature.....	166
4.5. Relationship between Specified and Measured Concrete Compressive Strength	170
4.6. Relationship between Calculated and Measured E_{ci}	173
4.7. Relationship between Measured and Predicted Variation of Modulus of Elasticity with Time	175
4.8. Relationship between Specified and Measured Concrete Unit Weight	176
4.9. Relationship between Specified and Measured Prestressing Force.....	177
4.10. Relationship between Specified and Measured Beam Length.....	178
4.11. Summary of Relationships between Measured and Specified/Calculated Properties ...	179

4.12. Solar Radiation Study	179
4.13. Data Collection Guidelines	182
CHAPTER 5 : Evaluation of Factors that Affect Prestressed Concrete Beam Camber and Deflection	184
5.1. Introduction.....	184
5.2. Sensitivity Analysis	184
5.2.1. Influence of Creep and Shrinkage Model on Pre-erection Camber	190
5.2.2. Influence of Model to Predict Modulus of Elasticity at Prestress Release and 28 Days	191
5.2.3. Influence of the Model for Capturing the Variation of Modulus with Time	193
5.2.4. Influence of Overstrength Factors	194
5.2.5. Influence of Time when Camber at Release is Measured.....	195
5.2.6. Slab and Screed Guidance	196
5.3. Summary and Conclusions	197
CHAPTER 6 : Evaluation of Factors that Affect Camber and Deflections in the Composite System	199
6.1. Introduction.....	199
6.2. Sensitivity Analysis	200
6.2.1. Influence of Creep and Shrinkage Model on Service Camber.....	205
6.2.2. Influence of Model to Predict Modulus of Elasticity at Prestress Release and 28 Days	206
6.2.3. Influence of Model for Capturing the Variation of Modulus with Time	208
6.2.4. Influence of Overstrength Factors	209
6.2.5. Influence of the Time when the Deck is Placed.....	210
6.2.6. Influence of the Temperature Gradient.....	211
6.3. Summary and Conclusions	213

CHAPTER 7 : Evaluation of Factors that Affect Beam Rebound and Behavior after Deck Replacement 216

7.1. Introduction.....	216
7.2. Comparison of Rebound and Net Camber after Deck Removal.....	216
7.3. Influence of Deck Replacement Time	219
7.4. Summary and Conclusions	220

CHAPTER 8 : Evaluation of Various Prediction Methodologies for Time-Dependent Flexural Deformation 221

8.1. Introduction.....	221
8.2. Evaluation of Prediction for Camber at Release.....	222
8.3. Evaluation of Predictions for Pre-erection Camber	225
8.4. Evaluation of Predicted Camber Growth Curve	227
8.5. Comparison of Incremental Time Step Method (Nilson 1987) and Proposed Prediction Methodology	235
8.5.1. Variation of Prestress and Prestress Losses with Time.....	236
8.5.1.1. Variation of Total Prestressing Stress with Time	236
8.5.1.2. Variation of Prestress Losses with Time	237
8.5.1.3. Comparison of Prestress Losses.....	238
8.5.2. Comparison of Camber Prediction.....	242
8.6. Comparison of Various Approaches.....	244
8.6.1. Comparison of Approaches No. 1 through No. 6 for Pre-erection Camber.....	245
8.6.2. Comparison of Approaches No. 2 to No. 6 for Service Camber without Deck Replacement.....	249
8.6.3. Comparison of Approaches No. 4 to No. 6 for Service Camber with Deck Replacement	255
8.6.4. Comparison of Approaches No. 1 to No. 6 for Stresses	256
8.6.4.1. Comparison of Effective Prestressing Strand Stress using Approaches No. 4 to No. 6.....	257

8.6.4.2. Comparison of Top and Bottom Fiber Stresses using Approaches No. 4 to No. 6	259
8.6.4.3. Comparison of Cross-sectional Stresses using Approaches No. 2 to No. 6	266
8.6.5. Impact of Method to Generate Time Steps within Approach No. 6	270
8.6.6. Influence of Number of Beam Sections Specified in Approach No. 6	271
8.7. Derivation of Time-Dependent Multiplier Method	272
8.8. Comparison of Proposed Prediction Methodology and Time-Dependent Multiplier Method	275
8.9. Summary and Conclusions	280
CHAPTER 9 : Summary and Conclusions.....	284
9.1. Chapter 3 - General:.....	284
9.2. Chapter 4 – Relationships between Specified and Measured Properties	285
9.3. Chapter 5 - Factors that Affect Prestressed Concrete Beam Camber	286
9.4. Chapter 6 - Factors that Affect Camber and Displacements in the Composite System ..	287
9.5. Chapter 7 - Factors that Affect Beam Rebound and Behavior after Deck Replacement.	288
9.6. Chapter 8 - Evaluation of Various Design Procedures	289
APPENDIX A : Previous Studies	294
A.1. Buettner and Libby (1979).....	294
A.2. Tadros et al. (1985).....	294
A.3. Kelly et al. (1987)	295
A.4. Brown (1998).....	295
A.5. Yazdani et al. (1999).....	296
A.6. Wyffels et al. (2000)	296
A.7. Jáuregui et al. (2002).....	297
A.8. Stallings, Barnes, and Eskildsen (2003)	297
A.9. Waldron (2004).....	298

A.10. Cook and Bloomquist (2005).....	298
A.11. Barr et al. (2005)	299
A.12. Hinkle (2006)	300
A.13. Rosa et al. (2007)	300
A.14. Jayaseelan and Russell (2007)	301
A.15. Omar et al. (2008)	301
A.16. Barr and Angomas (2010).....	302
A.17. Lee (2010).....	302
A.18. Tadros et al. (2011)	303
A.19. French and O'Neill (2012).....	303
A.20. Schrantz (2012)	304
A.21. Johnson (2012).....	304
A.22. Precast/Prestressed Concrete Institute Committee on Bridges (2012)	305
A.23. Bažant et al. (2012)	305
A.24. Storm et al. (2013)	306
A.25. Mahmood (2013)	306
A.26. He (2013)	307
A.27. Nervig (2014).....	308
A.28. Hofrichter (2014)	308
A.29. Isbilibiroglu (2014)	309
A.30. Nguyen (2014)	309
A.31. Honarvar et al. (2015)	310
A.32. Keraga (2016)	310
A.33. Menkulasi et al. (2018)	311
A.34. Mante et al. (2019).....	311

APPENDIX B : Prestressed Concrete Beam Camber Questionnaire.....	312
APPENDIX C : Creep and Shrinkage Models.....	339
C.1. Creep Coefficient Prediction Models.....	339
C.1.1. AASHTO LRFD 2020	339
C.1.2. ACI 209R-92	341
C.1.3. Bažant-Baweja B3	345
C.1.4. CEB MC90.....	348
C.1.5. CEB MC90-99.....	351
C.1.6. GL 2000.....	352
C.1.7. fib MC 2010	353
C.2. Shrinkage Strain Prediction Models.....	356
C.2.1. AASHTO LRFD 2020	356
C.2.2. ACI 209R-92	358
C.2.3. Bažant-Baweja B3	361
C.2.4. CEB MC90.....	363
C.2.5. CEB MC90-99.....	364
C.2.6. GL 2000.....	367
C.2.7. fib MC 2010	368
APPENDIX D : <i>MDOTCamber</i> Program User’s Manual	369
D.1. Tab 1: Introduction	371
D.2. Tab 2: Girder Properties.....	372
D.3. Tab 3: Girder Concrete Properties	377
D.4. Tab 4: Deck Properties.....	380
D.5. Tab 5: Prestressing Steel Properties.....	383
D.6. Tab 6: Creep and Shrinkage Model Selection	385

D.7. Tab 7: Time, Environment and Measured Camber Properties	386
D.8. Tab 8: Results	388
APPENDIX E : Example Problem for Proposed Prediction Methodology for Prestressed Concrete Girders.....	396
E.1. Definition of the Problem.....	397
E.2. Definition of Parameters	401
E.3. Camber at Release	430
E.4. Pre-erection Camber.....	435
E.5. Camber just after Placement of 1 st Deck	441
E.6. Camber just before Application of Barrier and Overlay Loads on 1 st Superstructure	443
E.7. Camber just after Application of Barrier and Overlay Loads on 1 st Superstructure	447
E.8. Camber just before Deck Removal Process	449
E.9. Camber just after Deck Removal Process	452
E.10. Camber just before 2 nd Deck Placement.....	455
E.11. Camber just after 2 nd Deck Placement	457
E.12. Camber just before Application of Barrier and Overlay Loads on 2 nd Super-structure	458
E.13. Camber just after Application of Barrier and Overlay Loads on 2 nd Super-structure...	460
E.14. Camber at Final Time.....	461
E.15. Nomenclature	466
APPENDIX F : Example Problem for Proposed Prediction Methodology for Steel Girders	473
F.1. Definition of the Problem	474
F.2. Definition of Parameters.....	478
F.3. Pre-erection Camber	490
F.4. Camber just after Placement of 1 st Deck	492

F.5. Camber just before Application of Barrier and Overlay Loads on 1 st Superstructure	493
F.6. Camber just after Application of Barrier and Overlay Loads on 1 st Superstructure	496
F.7. Camber just before Deck Removal Process	497
F.8. Camber just after Deck Removal Process	500
F.9. Camber just before 2 nd Deck Placement.....	503
F.10. Camber just after 2 nd Deck Placement.....	503
F.11. Camber just before Application of Barrier and Overlay Loads on 2 nd Superstructure..	504
F.12. Camber just after Application of Barrier and Overlay Loads on 2 nd Superstructure	505
F.13. Camber at Final Time	507
F.14. Nomenclature.....	510
APPENDIX G : Example Problem for Predicting Camber and Deflection for S11 Project using <i>MDOTCamber</i> Program	514
G.1. Problem Statement.....	515
G.2. Solution Steps	519
G.2.1. Tab 1: Introduction.....	519
G.2.2. Tab 2: Girder Properties.....	520
G.2.3. Tab 3: Girder Concrete Properties	523
G.2.4. Tab 4: Deck Properties.....	524
G.2.5. Tab 5: Prestressing Steel Properties.....	526
G.2.6. Tab 6: Creep and Shrinkage Model Selection	528
G.2.7. Tab 7: Time, Environment and Measured Camber Properties.....	528
G.3. Results - General.....	529
APPENDIX H : <i>MDOTCamber</i> v6.0 Installation Manual.....	534
REFERENCES.....	542
ABSTRACT.....	552
AUTOBIOGRAPHICAL STATEMENT	554

LIST OF TABLES

Table 2.1. MDOT multipliers.....	34
Table 2.2. PCI Multipliers (PCI Design Handbook 2011).....	34
Table 3.1. Description of approaches used to develop the proposed prediction methodology....	51
Table 3.2. Values used in the analysis for ageing coefficient and modulus of elasticity.....	79
Table 3.3. Approach used for creation of time array (for the cases with no deck replacement event)	109
Table 3.4. Parameter for aggregate type	128
Table 3.5. Comparison of modulus of elasticity prediction models.....	129
Table 3.6. Parameter for ACI 209R-92 model	130
Table 3.7. Coefficient s for strength class of cement based on fib MC 2010 model	132
Table 3.8. Concrete mixture properties for girder and deck	135
Table 4.1. Camber and displacement data sets.....	157
Table 4.2. Curing techniques used by precast beam fabricators that supply prestressed concrete bridge beams for MDOT	159
Table 4.3. Summary of the database obtained from literature	170
Table 4.4. Overstrength factors	173
Table 4.5. Relationship between calculated and measured modulus of elasticity at release, E_{ci} (calculated modulus is based on specified compressive strength).....	174
Table 4.6. Relationship between calculated and measured modulus of elasticity at release, E_{ci} (calculated modulus is based on measured compressive strength).....	175
Table 4.7. Comparison of specified and measured concrete unit weight.....	177
Table 4.8. Comparison of specified and measured prestressing force at release	178
Table 4.9. Comparison of specified and measured beam length.....	178
Table 4.10. Summarized results for the relationship between measured and specified parameters	179
Table 5.1. Influence of various parameters on camber at release	189

Table 5.2. Influence of various parameters on pre-erection camber	190
Table 6.1. Influence of various parameters on camber and displacements at service.....	204
Table 7.1. Comparison of rebound, net camber after deck removal, and net camber before new deck placement	218
Table 7.2. Comparison of net camber prior to placement of deck to net camber after removal of deck.....	219
Table 7.3. Influence of deck replacement time on beam rebound, net camber after deck removal, and net camber before new deck placement based on Approach No. 6 (S11 project)	220
Table 8.1. Summary of camber at prestress release predictions	224
Table 8.2. Summary of pre-erection camber predictions	227
Table 8.3. Comparison of separate and total prestress losses computed using the proposed prediction methodology and AASHTO LRFD 2020 Body provisions for the S11, B05, M57 and Shepherd Road-2 projects	242
Table 8.4. Comparison of several approaches for pre-erection camber.....	276
Table 8.5. Comparison of proposed multiplier with different initial camber values	277

LIST OF FIGURES

Figure 2.1. Stretched-wire method (Johnson 2012).....	12
Figure 2.2. Differential level instrument, tripod and surveying rod	13
Figure 2.3. Elastic-beam theory	17
Figure 2.4. Moment-area theorem.....	18
Figure 2.5. Moment-area theorem for self-weight only	21
Figure 2.6. Moment-area theorem for straight tendon profile	22
Figure 2.7. Moment-area theorem for harped tendon profile	23
Figure 2.8. Support conditions for conjugate beams (Hibbeler 2012).....	27
Figure 2.9. Tabulated equations for deflection calculations (PCI 2011)	31
Figure 2.10. Illustration of creep induced strain due to prestress and prestress loss	38
Figure 3.1. Adopted sign convention	55
Figure 3.2. Curvature diagrams for self-weight effect only	58
Figure 3.3. Curvature diagram for prestressing effect only	59
Figure 3.4. Implementation of Moment-area theorem for a typical prestressed concrete beam..	63
Figure 3.5. Distribution of forces in a prestressed concrete girder	69
Figure 3.6. Principle of superposition for increasing stress increments (Gilbert and Ranzi 2010)	70
Figure 3.7. Principle of superposition for decreasing stress increments (Gilbert and Ranzi 2010)	70
Figure 3.8. Effect of loading type on creep strain (Gilbert and Ranzi 2010).....	71
Figure 3.9. Representation of stress decrement with time (Gilbert and Ranzi 2010)	72
Figure 3.10. Illustration of age-adjusted effective modulus (AEMM) concept to capture creep effects.....	73
Figure 3.11. Illustration of Approach No. 1 for predicting time-dependent flexural deformations; a) assumed stress history, b) discretized stress history, and c) elastic and creep induced strains using Effective Modulus (EM) and Age-adjusted Effective Modulus (AAEM) approaches and based on discretized stress history	74

Figure 3.12. Distribution of forces in: a) composite prestressed concrete beam, and b) composite steel beam	82
Figure 3.13. Illustration of Approach No. 2 for predicting time-dependent flexural deformations; a) assumed stress history, b) discretized stress history, and c) elastic and creep induced strains due to discretized stress history.....	83
Figure 3.14. Curvature diagrams for composite and non-composite sections (Menkulasi 2018)	88
Figure 3.15. Illustration of Approach No. 3-5 for predicting time-dependent flexural deformations; a) assumed stress history, b) discretized stress history, and c) elastic and creep induced strains due to discretized stress history.....	91
Figure 3.16. Flowchart for Approach No. 5.....	100
Figure 3.17. Empirical model selection in the proposed prediction methodology	101
Figure 3.18. a) Illustration of effective modulus (EM) and age adjusted effective modulus (AAEM) concept, b) effect of loading age on creep coefficient according to AASHTO LRFD model for an interior prestressed concrete beam used in the S11 project, c) assumed stress history, idealized stress history, and creep induced strain due to a varying stress history (multiple time steps with multiple creep curves.....	105
Figure 3.19. Flowchart for Approach No. 6.....	125
Figure 3.20. Modulus of elasticity as a function of time based on the ACI 209R-92 Model	131
Figure 3.21. Modulus of Elasticity as a function of time based on the fib MC 2010 Model.....	133
Figure 3.22. Variation in creep coefficient and shrinkage strain based on different shrinkage and creep models for: a) PCBT 54 used in S11 project (Flange width 49 in., t_c = 1 day steam curing, t_{load} = 1 day, Cement Type I); b) Deck used in S11 project (t_c = 1 day steam curing, t_{load} =1 day, Type I cement, mixture proportions are based on a typical deck mix featuring normal weight concrete and fly ash)	137
Figure 3.23. Test setup for creep test	138
Figure 3.24. Sulfur capped concrete samples for creep and shrinkage tests.....	138
Figure 3.25. Measured temperature and ambient relative humidity in lab environment	139
Figure 3.26. Comparison of measured and predicted creep coefficient.....	140
Figure 3.27. Comparison of measured and predicted shrinkage strain.....	141
Figure 3.28. Temperature gradients based on AASHTO LRFD 2020 solar radiation zone III for: (a) Non-composite prestressed concrete beam (b) Composite prestressed concrete beam; (c) Steel beam alone; (d) Composite steel beam.....	147

Figure 4.1. Nominal dimensions (in mm) for CPCI-1600 prestressed concrete beams.....	161
Figure 4.2. a) Illustration of methodology for measuring camber in four CPCI-1600 prestressed concrete beams, b) photograph showing measurement setup and device made for taking consistent measurements across beam bottom flange, c) photograph showing device used to match the bottom flange of the beam and to receive the measuring rod.	164
Figure 4.3. a) Photograph showing the storage condition of box beams, b) Photograph showing the bottom of the girder where the measurements are taken relative to	165
Figure 4.4. 42” x 49” Bulb Tee Beams used in Saginaw Bridge Project.....	166
Figure 4.5. Ratio of measured over specified: a) initial concrete compressive strength, b) 28 day concrete compressive strength	172
Figure 4.6. Development of modulus of elasticity with time using specified (left) and measured (right) properties for: a) R04 (CG1); b) R04 (CG2); c) S02 (CG3); d) B02 (CG4) projects.....	176
Figure 4.7. Variation in camber due to radiation induced thermal gradient for: a) Girder G2-6, b) Girder G2-7, c) Girder G2-8, d) Girder G2-9	180
Figure 4.8. Variation in camber due to radiation induced thermal gradient for: a) Girder BL-1 and b) Girder BL-2	181
Figure 4.9. a) Variation in camber due to radiation induced thermal gradient, b) A view for the beams located near Wayne State University structure’s lab.....	182
Figure 5.1. a) Variation of pre-erection camber for S11 project using different creep and shrinkage models, and b) % Difference in the pre-erection camber for S11 project using different creep and shrinkage models with respect to AASHTO LRFD 2020 Body benchmark model	191
Figure 5.2. a) Comparison of calculated Modulus of Elasticity at Prestress Release and 28 days, and b) Influence of Model to Predict Modulus of Elasticity at Prestress Release and 28 Days for S11 project	193
Figure 5.3. a) Comparison of functions for predicting the development of modulus of elasticity with respect to time, and b) influence of considered time dependent modulus functions on camber for S11 project.....	194
Figure 5.4. a) Influence of inclusion of overstrength factors on camber for S11 project, and b) % Difference in camber with respect to no overstrength factors	195
Figure 5.5. Variation of predicted camber during the first 10 days after beam fabrication.....	196

Figure 5.6. Illustration of various stages of deflection during beam erection and deck placement for the S11 project	197
Figure 6.1. Influence of creep and shrinkage problems on the full displacement history of: a) prestressed concrete beams used in the S11 project, and b) steel beams used in the M20 project	206
Figure 6.2. a) Comparison of calculated modulus of elasticity at prestress release and 28 days based on different models, and b) influence of model to predict modulus of elasticity at prestress release and 28 Days on camber history for S11 project	208
Figure 6.3. a) Comparison of predictions for development of modulus of elasticity with respect to time based on different models, and b) influence of time dependent modulus model on camber for S11 project.....	209
Figure 6.4. Influence of inclusion of overstrength factors for S11 project.....	210
Figure 6.5. Influence of the time when the deck is placed on beam camber and displacements at service for: a) S10-2 project; b) S11 project.....	211
Figure 6.6. Influence of temperature gradients on beam camber and displacements at service: a) S10-2 project; b) S11 project, c) M20 project	213
Figure 8.1. Comparison of measured, predicted, and design camber at prestress release	224
Figure 8.2. Comparison of measured and predicted pre-erection camber	226
Figure 8.3. Variation of measured and predicted camber for the prestressed concrete beams fabricated in PSI Windsor for the Tiffin Street Overpass Project: a) full scale, b) reduced scale.....	229
Figure 8.4. Variation of measured and predicted camber using specified (left) and measured (right) concrete properties for the prestressed concrete beams provided by Hinkle (2006): a) Strand pattern I and b) Strand pattern II	231
Figure 8.5. Comparison of measured and predicted time dependent flexural deformations: a) b) c) d) e) Periodic composite prestressed concrete beam flexural deformation data by Branson et al. (1970); f) Periodic composite prestressed concrete beam flexural deformation data by Tadros et al. (1977); g) Composite steel beam flexural deformation data by Bradford and Gilbert (1991) and h) composite steel beam flexural deformation data by Fan et al. (2010).....	233
Figure 8.6. Influence of beam creep coefficient and beam shrinkage strain on beam camber considering the beams used in the Tiffin Street Overpass project.....	235
Figure 8.7. Variation of prestress as a function of time in a composite bridge system at midspan for S11, B05, M57 and Shepherd Road-2 projects	237

Figure 8.8. Variation of prestress losses with time in a composite bridge system at midspan for: a) S11 project; b) B05 project; c) M57 project and d) Shepherd Rd.-2 project	238
Figure 8.9. Comparison of calculated separate and total prestress losses using proposed prediction methodology and AASHTO LRFD 2020 Body provisions for: a) S11 project; b) B05 project; c) M57 project and d) Shepherd Rd.-2 project	241
Figure 8.10. Comparison of proposed method with the Incremental Time Step Method (Nilson 1987) for the: a) S11 project; b) B05 project; c) M57 project; d) Shepherd Rd.-2 project; e) M5 project, and f) S10-2 project	244
Figure 8.11. Comparison of various approaches for pre-erection camber for: a) S11 project, b) B05 project, c) M57 project and d) Shepherd Road-2 project.....	246
Figure 8.12. Comparison of various approaches using specified (left) and measured (right) concrete properties for: a) Tiffin Street Overpass Project, b) Cooper Road-1 Project (Hinkle 2006, Strand pattern I) and c) Cooper Road-2 Project (Hinkle 2006, Strand pattern II)	248
Figure 8.13. Comparison of predicted camber and displacement history based on Approaches No. 2 to No. 6 without deck replacement activity for: a) S11 project, b) B05 project, c) M57 project, d) Shepherd Road-2 project, e) S10-2 project and f) M20 project	251
Figure 8.14. Comparison of measured and predicted time dependent flexural deformations using Approaches No. 1-6: a) b) c) d) e) Periodic composite prestressed concrete beam flexural deformation data by Branson et al. (1970); f) Periodic composite prestressed concrete beam flexural deformation data by Tadros et al. (1977); g) h) Composite steel beam flexural deformation data by Bradford and Gilbert (1991) and i) j) composite steel beam flexural deformation data by Fan et al. (2010)	254
Figure 8.15. Comparison of predicted displacement history based on Iterations No. 4, 5, and 6 including a deck replacement activity after 50 years of service for: a) S11 project, b) B05 project, c) M57 project, d) Shepherd Road-2 project, e) S10-2 project, and f) M20 project	256
Figure 8.16. Comparison of strand stress histories using Approaches No. 4 to No. 6 for: a) S11 project, b) B05 project, c) M57 project, and d) Shepherd Road-2 project	259
Figure 8.17. Comparison of top and bottom fiber stress histories for the beams used in: a) S11 project, b) B05 project, c) M57 project, and d) Shepherd Road-2 project	263
Figure 8.18. Comparison of top and bottom fiber stress histories for the decks used in: a) S11 project, b) B05 project, c) M57 project, and d) Shepherd Road-2 project	266
Figure 8.19. Comparison of cross-sectional stress analysis at 100 days (left) and 75 years (right) for: a) S11 project, b) B05 project, c) M57 project, and d) Shepherd Road-2 project	269

Figure 8.20. Impact of method to generate the number of time steps on full camber and displacement history for: a) S11 project, b) S10-2 project.....	271
Figure 8.21. Sensitivity of camber predictions at midspan for the beams used in S11 project to the number of sections considered: a) number of odd sections, b) number of even sections, c) number of sections (odd or even)	272
Figure 8.22. Variation of multipliers as a function of time (prediction curves for each project are based on specified properties)	273
Figure 8.23. a) Influence of various parameters on pre-erection camber and b) Effect of $r(t)$ factor on the development of time-dependent multiplier equation	275
Figure 8.24. Camber growth prediction using proposed time-dependent multiplier method and specified (left) and measured (right) properties for: a) Tiffin Street Overpass Project, b) Cooper Road-1 Project (Hinkle 2006, Strand pattern I) and c) Cooper Road-2 Project (Hinkle 2006, Strand pattern II)	279

CHAPTER 1: Introduction

1.1. Background

The ride quality of bridges in Michigan has been the subject of increased focus in recent years. Two factors that can significantly impact ride quality are beam camber and deflection. Flexural deformations in composite bridge superstructures that feature prestressed concrete or steel beams with a cast in place concrete deck are time-dependent. Excessive camber growth due to concrete creep may result in an unpleasant transition from the roadway to the bridge and from bridge to the roadway. A similar problem may be caused by excessive downward deflections due to differential shrinkage between the deck and the girder. Time-dependent flexural deformations in such composite bridge superstructures combined with live load deflections may also affect the vertical under clearance, which may lead to traffic restrictions for the roadway underneath. The benefits of estimation of camber and deflection accurately in bridges can be summarized as follows:

- Improved ride quality
- Improved vertical clearance prediction
- Minimized construction delays
- Minimized extra costs
- Reduced design and/or functional modifications on the projects

Camber and deflection different than those estimated during design can also lead to the need for changes during construction, which can result in increased cost and longer construction duration. Differences between calculated and actual camber could result in either excessive or negative haunches. Excessive haunches may affect the embedment depth of extended stirrups in prestressed concrete beams or shear studs in steel beams. Negative haunches may impact deck

thickness over beam flanges. Similarly, any deviations from the specified deck surface elevations may result in grinding of the deck to ensure that the finished deck surface matches the vertical roadway geometry as close as possible.

A related concern is accurately predicting beam rebound and subsequent deflections when decks are removed for replacement. For this assessment, the “rule of thumb” procedure used by Michigan Department of Transportation, MDOT, designers have, in some cases, produced unreliable estimates of beam rebound and resultant camber. The consequences of inaccurate rebound prediction are similar to those of inaccurate pre-erection camber predictions (i.e. excessive or negative haunches). Steel beams should be able to rebound to their original position, assuming that the deterioration that has prompted the replacement of the deck has not affected the beams and that pin and roller supports behave as intended. Conversely, rebound in prestressed concrete beams is a function of the loading history and the competing effects of prestress and self-weight induced creep as well as differential shrinkage.

MDOT currently relies on multiplier based empirical equations to determine camber and long-term deflection for prestressed concrete beams. However, these expressions may not be suitable when concrete mixtures featuring high strength concrete or formulations different than those for which the multiplier method may be appropriate are specified for prestressed concrete beams. Therefore, there is a desire to re-evaluate existing MDOT procedures for estimating beam camber and deflection. Ideally, revised procedures should be validated by actual values for camber and deflection measured during Michigan bridge construction projects, as well as account for current fabrication practices.

The multiplier method used by many state DOTs to predict long-term camber was originally developed by Martin (1977) for precast double tees used in buildings with a 2 in.

concrete topping and normal strength concrete. However, the typical strength of concrete at prestress release has increased from about 4500 psi in the past to 6500 psi and even 12000 psi, as shown in recent work by FHWA and Nebraska Department of Roads (Morcous and Tadros 2009). This increase in concrete strength has led to complications when using the multiplier method, resulting in inaccurate camber predictions. For example, higher strength concretes are often used with more slender beams, which require a corresponding increase in prestress, and result in an expected level of camber greater than that associated with traditional mixes. While this increase in camber is somewhat counterbalanced by the greater modulus and lower creep associated with high strength concrete, it is uncertain what the extent of this counterbalancing effect is. Additionally, in higher strength concrete, the majority of creep and shrinkage take place in the first few months rather than throughout longer periods of time, as is the case for lower strength mixes, altering the assumption of gradual development of camber and deflection inherent in the multiplier method. The interaction of these factors is not only complex, but also consequential as they represent significant changes in the assumptions used to develop the multiplier method, causing it to produce inaccurate estimates of camber.

Another inherent problem associated with the multiplier approach is that the multipliers provide a single lump sum value to predict behavior, while camber and deflections are affected by many factors that vary with time such as creep, differential shrinkage, temperature gradients, age of concrete, loading history, etc. As a result, the use of a single numerical multiplier for predicting beam camber or deflections at a specific time is not an adequate approach to accurately capture the influence of the multiple factors that affect long-term beam response.

The research presented in this thesis aims to address the problem above, by providing a procedure in which the effects of all significant factors that influence camber and deflections are

quantified individually and appropriately combined to produce an accurate model for predicting camber, deflections, and rebound of MDOT beams as a function of time.

1.2. Research Objectives

The research objectives of this thesis are to:

1. Synthesize relevant research and current practices of other DOTs,
2. Explore methods for measuring camber,
3. Develop a procedure that can accurately predict the camber of prestressed concrete beams,
4. Develop a procedure that can accurately predict deflection at all stages of construction for all beam types, both new and existing,
5. Develop a procedure that can accurately predict rebound for existing beams during deck replacement.

1.3. Scope of Research

The final product of this research is a general framework that can be used to estimate beam camber, deflections, and rebound, at any point in time during the service life of the bridge. This is done by quantifying the influence of significant factors that affect camber and deflections at: 1) the material level through the use of appropriate models for predicting the free creep and shrinkage of concrete; 2) the cross-sectional level by determining how the distribution of strain, stress, and curvature is affected while maintaining strain compatibility, and; 3) the element and structure level by considering how changes in curvature at the cross-sectional level alter beam camber and deflection as a function of time. Furthermore, upper and lower bounds of predicted camber due to temperature induced effects are quantified according to AASHTO LRFD Bridge Design Specifications (2020). The developed guidelines will allow MDOT to improve slab and screed elevation calculations for future projects. Additional benefits include improved ride quality, more

precise camber and deflection predictions, and fewer adjustments in the field. Those impacted by this research include MDOT bridge designers, construction staff and consultants performing bridge work, as well as motorists.

1.4. Organization of Thesis

The thesis is composed of nine chapters and eight appendices.

CHAPTER 2 provides a review of the literature in the subject matter including the results of a nationwide survey. Moreover, methods for measuring camber and deflections and methods for predicting camber at release and long-term camber and deflections are provided.

CHAPTER 3 includes the proposed prediction methodology for short-term and long-term camber and deflection. Adopted sign convention, and assumptions made in this thesis are outlined. Furthermore, models for creep coefficient and shrinkage strain, modulus of elasticity and its development with respect to time are presented. In addition, formulae for estimating strand relaxation, elastic shortening, and temperature induced camber and deflection are shown.

CHAPTER 4 presents details about the beam camber and displacement dataset that was used to validate the proposed prediction methodology. Moreover, relationships between measured and specified parameters that affect camber and displacement are shown. A detailed explanation about the solar radiation study is also presented.

CHAPTER 5 presents an evaluation of factors that affect prestressed concrete beam camber through a sensitivity analysis, which was conducted using the proposed prediction methodology.

CHAPTER 6 presents an evaluation of factors that affect camber and displacements in composite bridge superstructures that feature prestressed concrete and steel beams through a sensitivity analysis that includes deck related parameters in addition to those considered for the beam.

CHAPTER 7 presents an evaluation of factors that affect beam rebound and behavior after deck replacement.

CHAPTER 8 presents an evaluation of various prediction methodologies for time-dependent flexural deformations. This includes various versions of the proposed prediction methodology, Incremental Time Step Method (Nilson 1987), a new time-dependent multiplier method, MDOT's fixed multiplier method and PCI's fixed multiplier method.

CHAPTER 9 provides a summary and conclusions.

APPENDIX A provides additional details on previous studies conducted on the subject matter.

APPENDIX B presents the details of the nationwide survey on the topic of beam camber and displacements.

APPENDIX C shows the formulae of creep and shrinkage models that are used in this thesis.

APPENDIX D illustrates the implementation of the proposed prediction methodology through a MATLAB based computer program called *MDOTCamber*.

APPENDIX E and APPENDIX F provide details of how the proposed prediction methodology is implemented to calculate camber and deflections in a prestressed concrete beam and steel beam superstructure, respectively.

APPENDIX G presents an example showing step-by-step procedure to conduct time-dependent analysis for S11 project using *MDOTCamber* program.

APPENDIX H presents a step-by-step procedure to install the *MDOTCamber* program.

CHAPTER 2: Literature Review

2.1. Introduction

Prestressed concrete beam camber and deflections is a topic that has been investigated in detail by various Department of Transportation (DOT)-sponsored research projects. Some of these include:

- North Carolina (Storm et al. 2013),
- Alabama (Stallings et al. 2003; Schrantz 2012; Johnson 2012; Isbilioğlu 2014; Mante 2016),
- Nebraska (Tadros et al. 2011),
- Washington (Rosa et al. 2007; Davidson 2014; Barr and Angomas 2010),
- Iowa (Nervig 2014; Honarvar et al. 2015; He 2013), Kentucky (Mahmood 2013),
- Texas (Kelly et al. 1987),
- Oklahoma (Jayaseelan and Russell 2007),
- Minnesota (French and O'Neill 2012; Wyffels et al. (2000)),
- Florida (Cook and Bloomquist 2005), and
- Idaho (Brown 1998).

Other studies include those conducted by Buettner and Libby (1979); Tadros et al. (1985); Hinkle (2006); Omar et al. (2008); Lee (2010); the PCI Committee on Bridges (2012); Keske (2014); and Hofrichter (2014).

Most of these studies have dealt with improving the accuracy of pre-erection camber. The subject of predicting long-term beam deflections in the composite system, and beam rebound once the deck is removed, has not received similar attention. A common conclusion found in the majority of these studies is that the main reasons for the difference between predicted and

measured camber are: 1) a disparity exists between the assumed and observed concrete properties (modulus, creep, shrinkage), and 2) local fabrication practices are often not properly considered within the prediction method such as type of curing, storage time and conditions, ambient relative humidity, temperature gradients, etc.

In past decades, NCHRP has sponsored related work such as NCHRP 496, in which guidelines were presented to obtain realistic estimates of concrete elastic modulus, shrinkage, and creep, and how these properties could affect prestress losses, camber, and deflections (Tadros et al. 2003). Much earlier, NCHRP Projects 12-1 and 12-6 addressed deflection and loss of camber in steel girders, considering shrinkage and creep in the concrete slab among other factors (Baldwin and Guell 1975), while the effect of thermal changes in concrete bridges was investigated by Imbsen et al. (1985) in NCHRP 276.

2.2. Summary of Previous Studies

The topic of accurately estimating time-dependent deflections in prestressed concrete bridge girders has received rigorous attention over the last 50 years. Since there are many parameters affecting time-dependent deflections of prestressed concrete bridge girders, it is not easy to review all related previous studies. Instead, a more recent and related studies are tried to be selected as shown in the APPENDIX A. Most studies reviewed the first step towards an accurate estimation of camber is the accurate estimate of the parameters that affect camber. For example, modulus of elasticity is a key factor that affects camber at release and long-term camber and is typically estimated using formulas that are a function of compressive strength, unit weight, and aggregate factors. Some previous studies investigated the modification of such formulas to account for regional practices, such as providing a calibration factor to account for local aggregates.

Similarly, a relationship between the specified and measured f'_c was investigated in previous studies to develop equations that relate the two. The establishment of such a relationship was conducted under the hypothesis that a more accurate input for the compressive strength would lead to a more accurate estimation of the modulus of elasticity. Also, many investigations on camber predictions were followed up with additional investigations for how to accurately predict, modulus, creep, and shrinkage properties of regionally used concrete mixes used in prestressed concrete beam fabrication. The accurate estimation of such properties is paramount because they serve as input values in the prediction framework. Many studies also pointed out the limitations of some of the existing creep and shrinkage models in terms of being able to predict how much concrete creeps and shrinks with time as well as the high variability between the available models. In many studies, it was emphasized that support conditions during storage also play a role in camber growth. Therefore, such storage conditions should be recorded including the type of support and should be taken into account when estimating pre-erection camber. The multiplier method resulted many times in inaccurate camber predictions due to its inability to account for a variety of conditions. Refinements of these methods have been somewhat successful in improving time-dependent deflection predictions, especially when the effects of creep and shrinkage are considered. However, the Incremental Time Step Method (Nilson 1987) was identified as one of the most accurate methods to predict camber.

2.3. Methods to Measure Deflection on Members

While most state DOTs have guidelines to measure camber, a consistent standard does not exist (Honarvar et al. 2015). Common methods of measurement in the precast industry include the use of simple, inexpensive tools such as a tape measure and a stretched string along the length of the beam, a rotary laser, and occasionally, survey equipment such as a theodolite or total station.

Different approaches also exist for where and when camber is measured; the top flange, bottom flange, and web have each been used in different instances, while immediate camber has been measured immediately upon release up to three hours after transfer (Hornavar et al. 2015).

Typically, the industry practice for quantifying camber involves measuring from the pallet to the bottom of the beam at mid-span using a conventional tape measure recorded to the nearest 1/8 in. It is required that the beam rests free on the pallet at the time of camber measurement. Variations in industry practice include differences in benchmark points for locating mid-span (where camber is measured); the refinement of the value read from the tape measure; and the time at which camber is measured. Potential sources of error in camber measurement include bed defects, friction between the precasting bed and beam ends, and inconsistently flat top or bottom flange surfaces locally or along the beam length.

The string method was used by Storm et al. (2013), and Menkulasi et al. (2014a, 2014b). However, caution should be exercised because this approach may lead to inaccurate measurements due to string elongation and anchorage slippage unless strict precautions are followed. Honarvar et al. (2015) used a rotary laser to accurately measure camber, as well as developed adjustment procedures for measuring camber using more traditional methods.

It was concluded that, regardless of the method used, camber can be measured accurately if appropriate procedures are followed, and possible errors are properly accounted for.

2.3.1. Tape Measure Method

This method can be employed mainly for measuring initial cambers of the prestressed concrete beams. Generally, before the beam is removed from precasting bed, a measurement is taken from precasting bed to the bottom of the beam. This value simply gives the initial camber at midspan. If desired, measurements for entire length can also be taken. However, this method may

give inaccurate results due to potential friction between beam and precasting bed, and bed deflections. Therefore, once the beam is moved from precasting bed to the temporary supports, the results may deviate.

2.3.2. Stretched-wire Method

Stretched-wire method is a manual deflection measurement system where a wire is mounted along the length of the girder and tensioned by using weights. Figure 2.1 shows a typical set up for the implementation of this method to measure the midspan camber with respect to center of bearings. Procedure for this method is as follows:

- I. Two holes are drilled into the web of the girder at the location where the measurements are wanted to be taken relative to. It may be at the far ends of the girder or at the bearing locations.
- II. Insert the bolts into the holes that have been drilled before.
- III. Tie the steel wire to one of the bolts and stretch it by using a pulley and weight which needs to be installed on the other bolt. It should be noted that the wire should not be tied to both bolts so that it can slide freely over the pulley. It is suggested to use a carabiner to install and remove the weight quickly, when necessary.
- IV. Position the precision scales on the web of the girder at the locations where camber measurements are wanted to be taken. It is suggested to place them on $1/4^{\text{th}}$, midspan and $3/4^{\text{th}}$ of the girder length.
- V. Take the first measurements at the locations where precision scales are installed. These values are used as a baseline. It should be noted that from step I to step V, steps should be done before prestressing strands are released.

- VI. After strands are released, take another set of measurements to determine the immediate camber. The difference between first and second measurements at the midspan simply gives the midspan camber.
- VII. For future camber measurements, repeat the step VI, and determine the camber by taking the difference between current measurements and measurements at step V (initial measurement which is used as a baseline.).

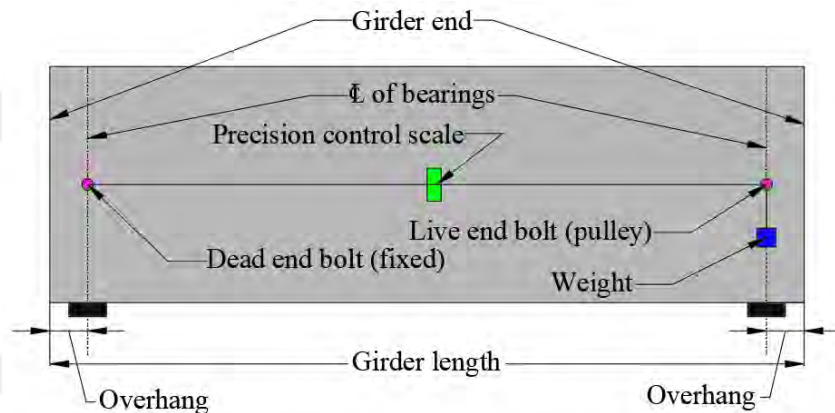


Figure 2.1. Stretched-wire method (Johnson 2012)

One of the biggest advantages of this method is that only one person is required to get the measurements. Besides, once the set up shown in Figure 2.1 is established, it is quite easy to take the measurements at any time. On the other hand, some error on the measurements may arise due to parallax. If there is a gap between wire and precision scales, depending on the angle that person looks to the precision scales, the measurements may deviate slightly. However, this error may be minimized by using mirrors behind the scale (Johnson 2012). Moreover, if the precasting bed is not perfectly level, that is precasting bed should be stiff enough in order not to deflect under girder self-weight effect, the initial (baseline) readings may not be a true reference for later measurements. Furthermore, if the measurements are taken outside, and the weather is windy, the steel wire may vibrate which may result in inaccurate readings. Last, but the greatest drawback of

the method, is that if for some reasons the precision scales are removed, the method fails since the method is entirely based on measurements relative to the baseline readings (Gross, 2000). Other than that, the method is simple and reliable.

2.3.3. Surveying Method

This method can be employed to measure camber and displacements at any age of the girder. Surveying rod with scale, tripod and a device for taking differential level such as theodolite, total station and laser level are the main necessary equipment to utilize this method, as shown in Figure 2.2. The process for this method basically involves taking elevation measurements of the beams at least three locations, at two ends and midspan, to determine the relative displacement of a location with respect to girder ends. It should be noted that since the method is based on relative elevations, while taking readings for different locations, the surveying rod must be positioned at the same level along the girder length. This could be bottom flange or top flange. Moreover, during a set of measurements, the position of instrument should not be changed.



Figure 2.2. Differential level instrument, tripod and surveying rod

Unlike stretched-wire method, this method requires at least two people: one holds the surveying rod, and the other takes the reading from instrument. A step-by-step procedure for this method is as follows:

1. Set up the instrument at a location where it can be used for entire beam locations without changing its location. Avoid placing it too close to girder because large horizontal change in the angles between the measurements might decrease the accuracy of the readings.
2. Take the first benchmark reading by choosing a place somewhere in the yard. This reading will be used to ensure the level of the instrument is not changed during taking measurement.
3. Mark the locations where the readings are wanted to be taken for. For comparing the displacement of a location relative to girder ends, at least three locations must be marked: two for girder ends, and one for midspan.
4. Select a place where the surveying rod is positioned while taking measurements along the girder. This may be bottom flange or top flange.
5. Position the surveying rod at an end of the girder and take the first reading from the instrument. It should be noted that while taking readings, the rod must be perfectly vertical.
6. Repeat step 5 for other end of the girder and other locations where measurements are wanted to be taken.
7. Take the final benchmark reading at the same location in step 2. Compare the benchmark recording and be sure that they are same. If not, the procedure should be repeated.
8. Using recorded elevations and similar triangle rule, determine the camber at desired locations.

Like other methods, potential errors arise from the persons who takes the measurements and holds the surveying rod. Moreover, the places where the surveying rod is positioned may not

be fully identical throughout the girder length. Thus, this may cause inaccurate results. On the other hand, although equipment costs are higher than stretched-wire method, surveying method is relatively common for determining beam deflection. Besides, in this thesis, some site trips have been conducted to determine the effect of solar radiation of beam deflection as well as determining midspan camber of some beams using this method. Detailed explanations how often and how measurements are taken are presented in section 4.12.

2.3.4. Photogrammetry

Unlike the methods mentioned above, photogrammetric surveying is rarely used by people from precast industry. The method enables three-dimensional measurements from two-dimensional photographs taken of an object. Basically, the procedure to be followed while using this model taking at least two pictures of an object with target points from different camera positions. The points on the objects provides a line of sight on the object of the perspective center of the camera. Then, using similar triangle technique, the elevation differences for required locations can be obtained. This method may be preferred for the projects where using aforementioned methods cannot be used due to environmental conditions. Otherwise, it is not used by engineers because of its limited use, equipment requirement, high cost and time limitations.

Jauregui et al. (2003) conducted a study related to measuring vertical bridge deflection using photogrammetry. One laboratory and two field exercises were performed and results obtained from photogrammetry, level road, total station readings and elastic finite element analysis results were compared. The study concluded that results from photogrammetry agrees well with the other methods.

2.4. Methods to Predict Camber at Release (Initial Camber)

There are several methods to predict camber at release for prestressed concrete beams such as moment-area theorems, direct integration method, conjugate-beam method, virtual work method, and tabulated equations. Among these, due to its simplicity, moment-area theorems are commonly used to predict the camber at release. It is also adopted in this thesis to calculate the predicted camber and deflections along the beam span. Conjugate beam method is also easy-to-use but the consideration of support conditions for conjugate beam may not be straightforward. Similarly, direct integration method is another robust approach to estimate the deflection of members, but integration constants may make the process overwhelming, especially when piecewise moment equations are needed to be used. On the other hand, due to its complexity, virtual work method is not preferred for estimating camber at release for prestressed concrete elements. Furthermore, tabulated camber equations are easy-to-use and cover most of the prestressing strand layouts. In this section of the thesis, each of these techniques are summarized.

2.4.1. Moment-Area Theorems with Elastic-Beam Theory

The moment-area method for computing slopes and deflections of beams was developed by Charles in 1873. (Kassimali 1999). The method is basically based on two theorems in which the geometry of elastic curve of a beam is related to its curvature diagram. Since the method provides semi-graphical solution for calculating the beam deflection due to bending, it is more convenient when compared to other methods. Besides, since the procedure requires graphical representation, following the adopted sign convention in this method is crucial. It is assumed that positive slope angle, θ , is measured counterclockwise from horizontal axis, and positive deflection is upward. It should be noted that this sign convention is only applicable for this part. The sign convention adopted in this thesis is shown in Figure 3.1.

When a beam is subjected to bending moment, the fibers along the cross-sectional height are deformed. By assuming plane sections before bending remain plane after bending, there is an angle occurring between two sections along the span, as shown in Figure 2.3. If the material is assumed to be homogeneous, and behaves linearly elastic, the following equation can be obtained.

$$\frac{1}{\rho} = \frac{M}{EI} \quad 2-1$$

Where:

ρ is the radius of curvature at a specific point on the elastic curve,

M is the bending moment in the beam at the point where ρ is determined,

E is the modulus of elasticity,

I is the moment of inertia at the section considered.

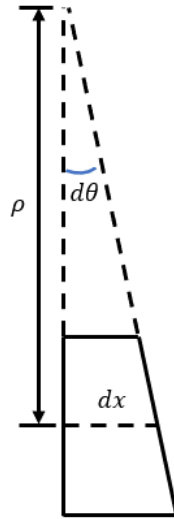


Figure 2.3. Elastic-beam theory

From small angle assumption, it can be inferred from Figure 2.3 that $\rho d\theta = dx$.

Combining this with Eqn. 2-1, Eqn. 2-2 is obtained.

$$d\theta = \frac{M}{EI} dx \quad 2-2$$

If both sides of the Eqn. 2-2 are integrated, it can be seen that the change in slope between two points on the elastic curve equals the area of M/EI diagram between these two points, as shown in Figure 2.4 (Hibbeler 2011). This is the first theorem of the moment-area method. On the other hand, the second theorem of this method states that the vertical deviation between tangent at a point (B in Figure 2.4) and tangent extended from other point (A in Figure 2.4) equals to the first moment of the area between these two points with respect to the point where the vertical deviation is measured from (B in Figure 2.4).

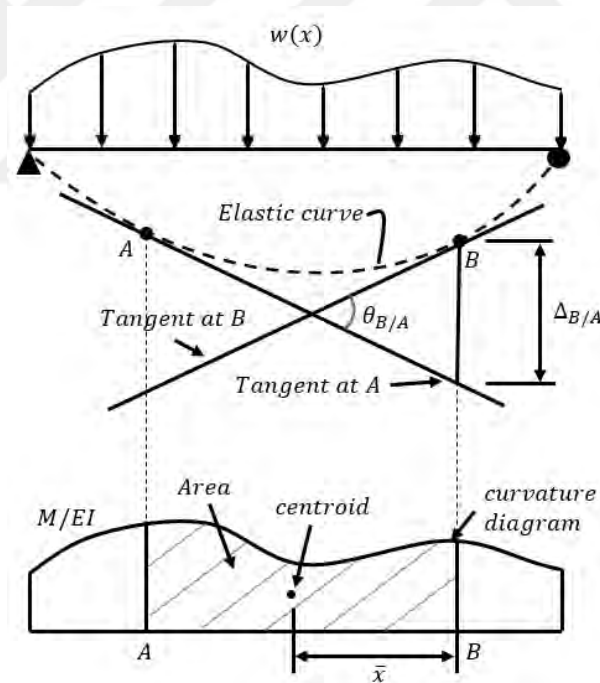


Figure 2.4. Moment-area theorem

Where:

$\theta_{B/A}$ is the angle of the tangent at B measured with respect to the tangent at A (radians).

$\Delta_{B/A}$ is the vertical deviation between point B on the elastic curve and extended tangent from point A,

$w(x)$ is the arbitrary load on the member,

M/EI is the curvature, (M is the moment and EI is the flexural rigidity of the member),

$\bar{x}$ is the centroid of the area between points A and B with respect to the point B.

Therefore, based on the theorems stated above, $\theta_{B/A}$ and $\Delta_{B/A}$ can be expressed as follows:

$$\theta_{B/A} = \int_A^B \frac{M}{EI} dx \quad 2-3$$

$$\Delta_{B/A} = \bar{x} \int_A^B \frac{M}{EI} dx \quad 2-4$$

As it can be inferred from the explanations above, this method provides really convenient solutions if the curvature diagram can be easily drawn. For the statically determinate structures, such as simply-supported beams, the curvature diagram, hence the elastic curve can be easily drawn. Hence, the moment-area method is handy to use for simply-supported prestressed concrete or steel beams with symmetric loading conditions because the vertical deviation between a point at the end of the girder (assuming no overhang) and at the midspan gives midspan deflection directly. Some examples about the implementation of this method for prestressed concrete beams are given below. On the other hand, for statically indeterminate structures which most of the engineering structures are, computer programs may be inevitable because moment diagram, hence the curvature diagram and elastic curve of indeterminate structures may not be easily determined. In this case, it may be a quite time-consuming process to apply this method to determine the

deflection of a member. However the scope of this thesis is to develop a procedure to predict camber and deflections for statically determinate structures, rather than indeterminate ones.

A simply supported beam under its self-weight effect only is shown in Figure 2.5. Since the vertical deflection at the girder end is zero, $\Delta_{A/B}$ means the midspan deflection, and it is obtained by multiplying the shaded area with its centroid. From geometry, the shaded area equals to $(2/3)bh$ where b is the base and h is the height of the parabola. In this case, $b = L/2$ and $h = wL^2/8EI$, which implies that shaded area equals $wL^3/(24EI)$. Similarly, the centroid of the parabola with respect to the point A as shown in Figure 2.5 equals to $5b/8$, which implies $\bar{x} = 5L/16$. Therefore, the vertical deviation between point A and B, which means midspan deflection, can be calculated by Eqns. 2-5 and 2-6. Eqn. 2-6 is the well-known formula for deflection of a simply-supported beam under uniform load conditions.

$$\Delta_{A/B} = A\bar{x} \quad 2-5$$

$$\Delta_{A/B} = \left(\frac{wL^3}{24EI} \right) \left(\frac{5L}{16} \right) = \frac{5wL^4}{384EI} \quad 2-6$$

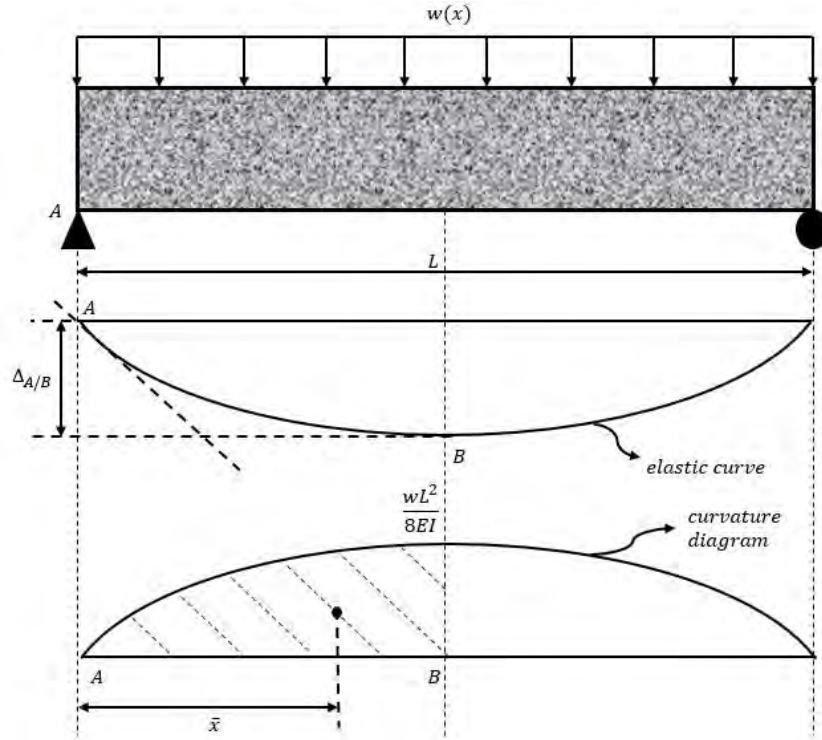


Figure 2.5. Moment-area theorem for self-weight only

For eccentrically applied straight prestressed tendon profile, as shown in Figure 2.6, the deflection can be calculated using by multiplying the area of shaded area with its centroid. Since the prestressing force is eccentric, the applied moment due to tendons are constant and equal to $-Pe$, if transfer length is ignored, where P is the effective force on the strands and e is the eccentricity. Hence, the first moment of the shaded area with respect to point A can be calculated by Eqn. 2-7.

$$\Delta_{A/B} = -\left(\frac{Pe}{EI}\right)\left(\frac{L}{2}\right)\left(\frac{L}{4}\right) = -\frac{PeL^2}{8EI} \quad 2-7$$

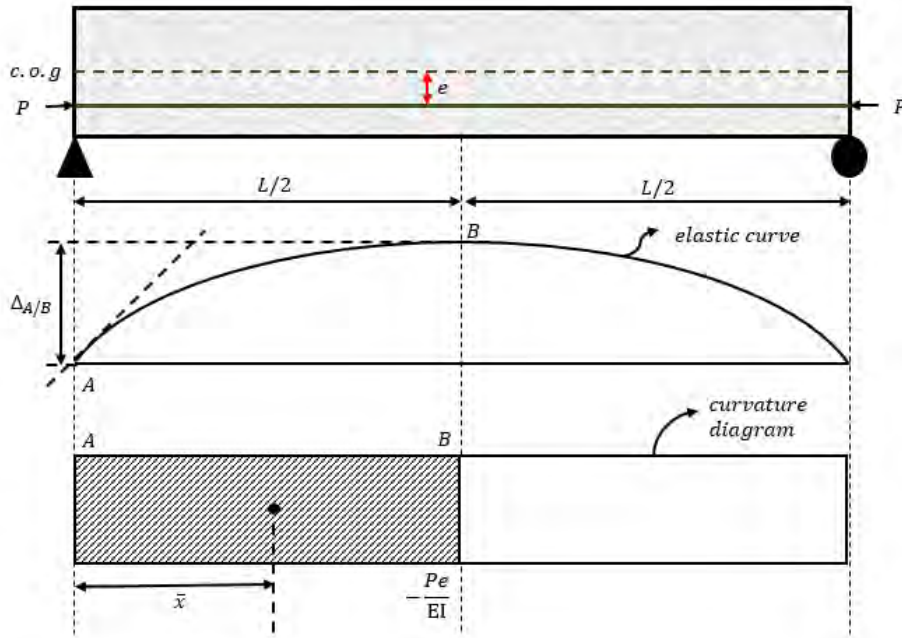


Figure 2.6. Moment-area theorem for straight tendon profile

For prestress layout which has harped tendons as shown in Figure 2.7, which is also common practice in prestressing industry to decrease the tension stresses at the ends, the curvature diagram can be divided into several pieces to facilitate the deflection computations. Then, the first moments of each piece are calculated with respect to the point A. Eqn. 2-9 may seem to be more complex than Eqns. 2-6 and 2-7, however the it is obtained following the same procedure.

$$\Delta_{A/B} = \sum_{i=1}^{i=3} A_i \bar{x}_i \quad 2-8$$

$$\Delta_{A/B} = -\left(\frac{Pe_e}{EI}\right)(aL)\left(\frac{aL}{2}\right) - \left[\frac{P(e_m - e_e)}{EI}\right]\left(\frac{aL}{2}\right)\left(\frac{2aL}{3}\right) - \left(\frac{Pe_m}{EI}\right)\left(\frac{L}{2} - aL\right)\left(\frac{0.5L - aL}{2}\right) \quad 2-9$$

If Eqn. 2-9 is rearranged, Eqn. 2-10 is obtained.

$$\Delta_{A/B} = -\frac{PL^2}{8EI} \left[e_m + (e_e - e_m) \frac{4a^2}{3} \right] \quad 2-10$$

Where:

e_m and e_e are midspan and end eccentricities, respectively,

P is the effective prestressing force in the tendons,

EI is the flexural stiffness of the beam, (E is elastic modulus and I is moment of inertia),

L is the girder length,

a is a coefficient for hold-down location (aL gives the location of harping point).

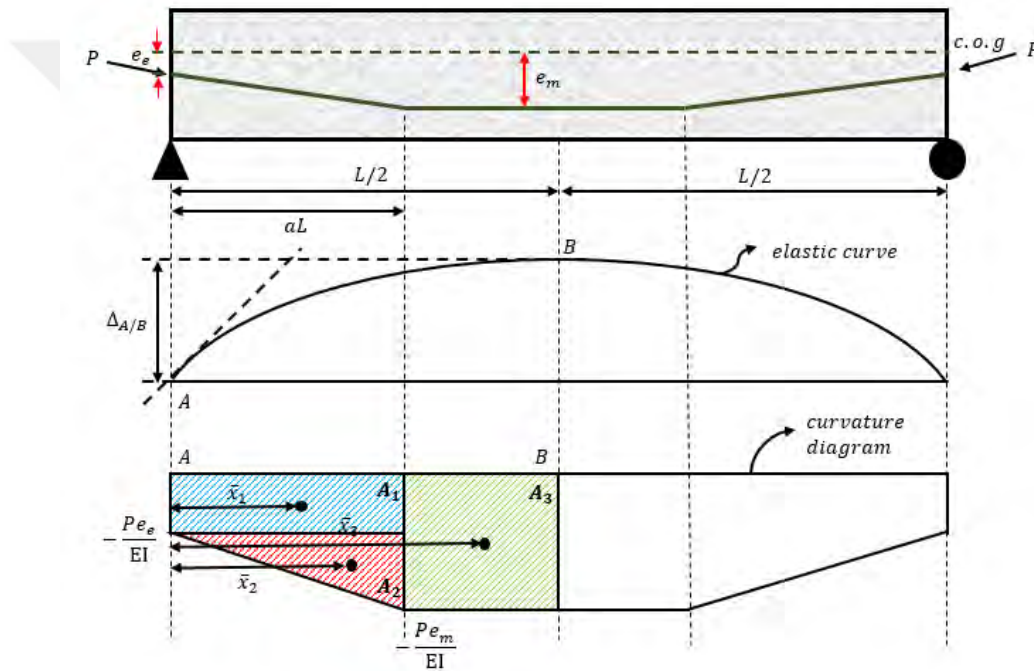


Figure 2.7. Moment-area theorem for harped tendon profile

As it can be understood from the examples given above, the deflection of the simple supported beams with prestressing tendons can be calculated using moment-area theorem with elastic-beam theory. This is because of the straight line nature of the curvature diagrams of such beams. However, when debonding and transfer lengths are involved, and if there is overhang length (i.e. the supports are not located at the far ends of the girder), it may be a time-consuming process to employ this method by hand to determine the camber and deflection at midspan simply because of constructing the actual moment diagram along the span. In fact, if deflections other

than midspan are wanted to be calculated, the process may be much more time-consuming. In that case, utilization of a tool such as Excel, Mathcad and MATLAB is highly suggested to facilitate the computational effort and to minimize the possible calculation errors.

So far, the calculation of midspan deflection has been discussed. However, the deflection at any other point apart from midspan can also be computed using this method. In this case, the deflections at locations other than midspan may not be calculated directly because the vertical deviation between a point at the end and a point at any location other than midspan would not give the deflection at considered location. Therefore, some geometrical expressions are needed to be derived. Detailed procedure about how deflections can be computed throughout the span is discussed in section 3.4.2.

It has been stated that the programs named *ALCAMBER* by Mante et al. (2016), *PGSuper* by WsDOT, *PSBEAM* by Eriksson Technologies, and *LEAP Conspan* by Bentley Systems are utilizing this method to predict the short-term deflections for flexural members (Mante 2016). This shows that this method can easily be programmed by considering any type of loading, i.e. distributed or point loads, and any strand layout including harped and debonded strands. Similar to other tools, the one created in this study, named as *MDOTCamber*, also utilizes this method to compute the camber and displacements at any location along the span. Detailed information about how this method is utilized in the *MDOTCamber* program is provided in section 3.4.2.

2.4.2. Direct Integration Method

This method essentially involves expressing the curvature diagram along the span as a closed form equation and integrating it subsequently to obtain the expression for deflected shape of the member. It is shown in the Figure 2.3 that the radius of curvature can be expressed as Eqn.

2-1. If x is axis for the span, and y is axis for the deflection, Eqn. 2-1 can be expressed as Eqn. 2-11 by using cartesian coordinates from calculus.

$$\frac{1}{\rho} = \frac{d^2y/dx^2}{[1 + (dy/dx)^2]^{3/2}} \quad 2-11$$

Where:

x and y are the position along the span and vertical deflection, respectively.

Since the deflections are quite small when compared to beam length, the term $(dy/dx)^2$ is negligible in comparison with unity. Therefore, Eqn. 2-11 can be rewritten as:

$$\frac{1}{\rho} = \frac{d^2y}{dx^2} \quad 2-12$$

If Eqns. 2-1 and 2-12 are combined, the following second order differential equation, known as Bernoulli-Euler beam equation is obtained.

$$\frac{d^2y}{dx^2} = \frac{M}{EI} \quad 2-13$$

Similarly, if Eqns. 2-2 and 2-12 are used together, the following first order differential equation is obtained.

$$\frac{d\theta}{dx} = \frac{M}{EI} \quad 2-14$$

If the expression in Eqn. 2-14 is integrated once, an equation for slope along the span is obtained. Similarly, if Eqn. 2-13 is integrated twice, an equation for deflection along the span is obtained. During integration, integration constants should be taken into account. They can be calculated by using the boundary conditions.

The direct integration method is one of the most convenient approaches to determine slopes and deflections of a member if the curvature can be expressed as a single continuous function along the span. However, the application of this method could be overwhelming for the structures where curvature function is not continuous because due to discontinuity in the curvature equation, additional integration constants come out which need to be determined by using continuity of the elastic curve. For example, a prestressed concrete beam with debonded and harped strands, curvature function cannot be defined as one continuous function, but can only be defined using several piecewise functions. In addition to this, considering other factors that affect camber and deflection of the girder, such as transfer length and location of support conditions while defining the moment, hence the curvature function, makes the process cumbersome. Even if the piecewise functions for curvature are somehow obtained, calculations of the integration constants at each location where discontinuity exists in the curvature expression would be a time-consuming process. However, the drawback of this method may partly be circumvented by utilizing the singularity functions.

2.4.3. Conjugate-Beam Method

This method, developed by Mohr in 1868, provides a procedure similar to moment-area method to compute the slopes and deflections of beams. One of the differences of this method from moment-area method is that drawing an elastic curve is not required. This method relates the curvature of a beam to its slope and deflection by considering a conjugate beam. A conjugate beam is a fictitious beam such that it has the same length as the real beam has, but it is externally supported and internally connected in a way that when it is loaded with curvature diagram values, M/EI , of a real beam, its shear and moment diagram give slope and deflection in real beam at a considered location, respectively. One of the most important things while determining the

conjugate beam is that the shear and bending moment diagram at any point on the conjugate beam must be consistent with slope and deflection at that point on the real beam. For instance, if end support condition in the real beam is fixed, corresponding end condition on the conjugate beam should be free end because there cannot develop any slope and deflection at the fixed end, so there should not be any shear and moment on the conjugate beam at corresponding end location. In Figure 2.8, possible support conditions in real beam and their versions for conjugate beam are presented.

Real Beam			Conjugate Beam		
1)	θ $\Delta = 0$	 pin	V $M = 0$	 pin	
2)	θ $\Delta = 0$	 roller	V $M = 0$	 roller	
3)	$\theta = 0$ $\Delta = 0$	 fixed	$V = 0$ $M = 0$	 free	
4)	θ Δ	 free	V M	 fixed	
5)	θ $\Delta = 0$	 internal pin	V $M = 0$	 hinge	
6)	θ $\Delta = 0$	 internal roller	V $M = 0$	 hinge	
7)	θ Δ	 hinge	V M	 internal roller	

Figure 2.8. Support conditions for conjugate beams (Hibbeler 2012)

As explained above, after determining the curvature diagram and conjugate beam, slope and deflection at any point can be calculated easily. In fact, Kassimali (1999) states that conjugate-beam method provides a more convenient solutions for slopes and deflections of a member than what moment-area method does. However, it is not preferred in this thesis while predicting camber and deflections along the span.

2.4.4. Virtual Work Method

Unlike other methods, virtual work method, developed by Bernoulli in 1717, provides a solution to compute the slope and deflection of members which have complicated load configurations. To find a work done by a force, the displacement along the direction of force is simply multiplied by that force. Principle of virtual, i.e. imaginary, work states that if a deformable structure is subjected to any small deformation, the virtual external work done by the virtual external forces acting through the real external displacements and rotations is equal to the virtual internal work done by the virtual internal forces acting through the real internal displacements and rotations. (Kassimali 1999). This simply means that the work caused by virtual external force is equal to the work caused by virtual internal forces.

To exemplify the usage of this method, consider a simply supported uniformly loaded beam. If the midspan deflection is to be calculated, a unit virtual load is applied at that location along the direction of displacement. Assuming under this virtual load the beam is deflected by an amount of Δ . Therefore, the work caused by external virtual load can be calculated by using Eqn. 2-15.

$$W_{ext} = 1(\Delta) \quad 2-15$$

Where:

W_{ext} is the external work,

Δ is the displacement caused by virtual load.

Moreover, applied virtual external load causes also internal displacements along the span. For each infinitesimal element on the beam along the span, the internal work can be expressed as Eqn. 2-16.

$$dW_{in} = M_v(d\theta) \quad 2-16$$

Where:

W_{in} is the internal work,

M_v is the virtual internal moment,

θ is the rotation caused by virtual internal bending moment on a particular element on the member.

Combining Eqns. 2-2 and 2-16, Eqn. 2-17 is obtained as follows:

$$dW_{in} = M_v \left(\frac{M}{EI} \right) dx \quad 2-17$$

Where:

M is the bending moment due to real loading.

EI is the flexural stiffness of the member, E is the elastic modulus and I is the moment of inertia.

Therefore, total work caused by virtual internal load is calculated by integrating Eqn. 2-17 along the span.

$$W_{in} = \int_0^L M_v \left(\frac{M}{EI} \right) dx \quad 2-18$$

By equating the external work (Eqn. 2-15) to the virtual internal work (Eqn. 2-18), Eqn. 2-19 is obtained as follows:

$$\Delta = \int_0^L M_v \left(\frac{M}{EI} \right) dx \quad 2-19$$

Similarly, the slope at any point on the member can be found like in Eqn. 2-19, however, the virtual external moment, in this case, is the moment due to unit moment at that location. As can be seen above, although the method may provide convenient solution procedure for relatively complex structures, it is not preferred due to its complexity. Since the scope of the thesis is limited to the calculation of camber and deflections of simply supported prestressed concrete or steel beams, it is a bit too advanced method to employ.

2.4.5. Tabulated Equations

Another simple but quite efficient method for calculating camber at release at midspan is using the tabulated equations. PCI (2011) provided most of the possible strand layouts and corresponding deflection equations to calculate the camber at midspan due to prestressing effect only (Figure 2.9). For the cases where strand layout is not one of the layouts provided in Figure 2.9, several strand patterns may be superpositioned to get the desired strand layout. Besides, the deflection caused by girder self-weight can be easily calculated using Eqn. 2-6. By summing the camber caused by prestressing effect and deflection caused by girder self-weight, net camber at release can be calculated.

It should be noted that these equations (Figure 2.9) do not take transfer length into account, and they can provide the camber caused by prestressing at midspan only. In other words, the

deflected shape of the beam along the span cannot be determined using such equations. Moreover, for the cases where straight, harped and debonded strand layout exist, employing superposition method with corresponding tabulated equation may be tedious. In any case, it is good to be aware of these equations below for checking the program outputs.

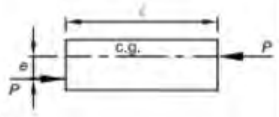
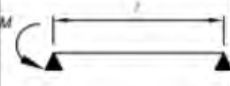
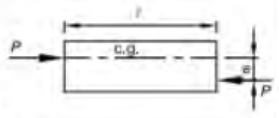
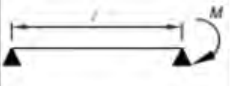
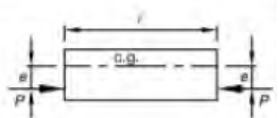
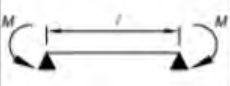
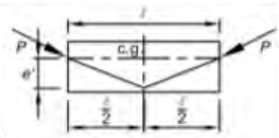
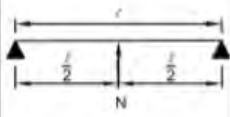
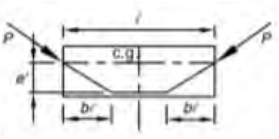
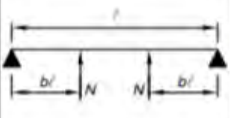
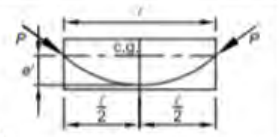
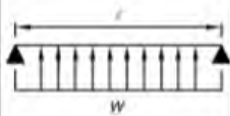
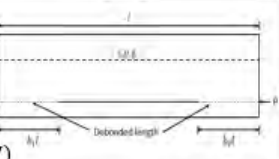
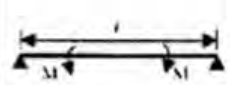
PRESTRESS PATTERN	EQUIV. MOMENT OR LOAD	EQUIVALENT LOADING	CAMBER	END ROTATION	
					
(1) 	$M = Pe$		$+\frac{Ml^2}{16EI}$	$+\frac{Ml}{3EI}$	$-\frac{Ml}{6EI}$
(2) 	$M = Pe$		$+\frac{Ml^2}{16EI}$	$+\frac{Ml}{6EI}$	$-\frac{Ml}{3EI}$
(3) 	$M = Pe$		$+\frac{Ml^2}{8EI}$	$\frac{Ml}{2EI}$	$-\frac{Ml}{2EI}$
(4) 	$N = \frac{4Pe'}{l}$		$+\frac{Nl^3}{48EI}$	$\frac{Nl^2}{16EI}$	$-\frac{Nl^2}{16EI}$
(5) 	$N = \frac{Pe'}{b'l}$		$\frac{b(3-4b^2)Nl^3}{24EI}$	$\frac{b(1-b)Nl^2}{2EI}$	$\frac{b(1-b)Nl^2}{2EI}$
(6) 	$w = \frac{8Pe'}{l^2}$		$\frac{5wl^4}{384EI}$	$\frac{wl^3}{24EI}$	$-\frac{wl^3}{24EI}$
(7) 	$M = Pe$		$\frac{Ml^2}{8EI} \left(\frac{1-2b_1^2-2b_2^2}{1-2b_1} \right)$	$\frac{Ml^2}{2EI} \left[\frac{(1-2b_1)^2-b_2^2}{1-2b_1} \right]$	$-\frac{Ml^2}{2EI} \left[\frac{(1-2b_1)^2-b_2^2}{1-2b_1} \right]$

Figure 2.9. Tabulated equations for deflection calculations (PCI 2011)

2.5. Methods to Predict Long-term Deflections

Unlike methods mentioned for predictions for camber at release, the computation of long-term camber and deflection is more complex because of the number of parameters involved and the time-dependent and interrelated nature of these parameters. It is impossible to eliminate the error caused by such parameters, but reasonable accuracy can be obtained if a good prediction method is employed. Although AASHTO LRFD (2020) provides limited guidance for long-term camber calculation, ACI 435R-95 (1963; 2003) summarizes seven methods for computing long-term deflections in prestressed concrete one-way flexural members, which include:

- 1) Multiplier method (Martin 1977),
- 2) Incremental time-step method (Nilson 1987),
- 3) Axial Strain and curvature method (Ghali and Favre 1986),
- 4) Approximate time-step method (Branson and Ozell 1961),
- 5) Prestress loss method (Tadros et al. 1985),
- 6) CEB-FIB Model Code method (CEB-FIP 1990), and
- 7) Section curvature method (ACI 435R-95).

Various modified forms of these basic methods have been presented by different standards and guidelines, with attempts to refine results for a particular application of interest. In this report, the Time-dependent Cross-sectional Curvature Method is proposed for prediction of long-term camber and displacements. Besides, multiplier methods (PCI and MDOT multipliers) and Incremental Time Step Method (Nilson 1987) are also presented for comparison purposes. Although remaining methods have not been employed, they are briefly explained in this part of the thesis. APPENDIX A provides additional details about various studies conducted on the subject of camber prediction. CHAPTER 3 shows the detailed information about proposed

methodology. Moreover, a comprehensive discussion about how this method is implemented in this thesis is presented in section 3.5.

2.5.1. Multiplier Method

The multiplier method was originally developed by Martin (1977) for precast prestressed concrete members with a 2 in. concrete topping and normal strength concrete. It is the simplest method among the methods for estimating long-term deflections because the short-term deflection computations are amplified by using a couple of multipliers. This means that after the short-term calculations are performed, long-term deflections can be estimated without requiring so much calculation effort. However, this simplicity may sometimes cause unreliable estimations since creep, shrinkage and prestress relaxation effects are not quantified specifically for a project. Especially for bridge girder with high strength concrete subject to a variety of environmental conditions, the method may result in significantly high camber estimations.

The multiplier method was later refined by Zia (1979) and Tadros (1985) and was ultimately adopted in the PCI Design Handbook (2010) and the PCI Bridge Design Manual (2011) (Table 2.2). However, Stallings et al. (2003) concluded that the PCI Design Handbook (2010) multiplier method significantly overestimated camber at the time of girder erection. In 2011, Tadros et al. (2011) revisited the topic of precast girder camber variability and concluded that designs should allow for up to a 50% variation in results unless future research offers a refined procedure.

Moreover, there are some DOTs which modify the proposed multipliers stated in PCI Design Handbook (2010) based on their experiences about variation in long-term deflections. For example, Michigan Department of Transportation (MDOT), uses multipliers that are not constant but depends on the cross-sectional properties of beam, as shown in Table 2.1.

Table 2.1. MDOT multipliers

MDOT Multipliers	
Factor applied to prestressing force at release	$1.9 + 0.6(I_{girder}/I_{composite})$
Factor applied to beam self-weight at release	$2.1 + 0.7(I_{girder}/I_{composite})$
Factor applied to slab when poured (including such as SIP forms, diaphragms, and utility loads)	$1.0 + 1.1(I_{girder}/I_{composite})$
Factor applied to barrier and sidewalk when poured	2.3

Table 2.2. PCI Multipliers (PCI Design Handbook 2011)

	Without composite topping	With composite topping
At erection:		
(1) Deflection (downward) component—apply to the elastic deflection due to the component weight at release of prestress	1.85	1.85
(2) Camber (upward) component—apply to the elastic camber due to prestress at the time of release of prestress	1.80	1.80
Final:		
(3) Deflection (downward) component—apply to the elastic deflection due to the component weight at release of prestress	2.70	2.40
(4) Camber (upward) component—apply to the elastic camber due to prestress at the time of release of prestress	2.45	2.20
(5) Deflection (downward)—apply to elastic deflection due to superimposed dead load only	3.00	3.00
(6) Deflection (downward)—apply to elastic deflection caused by the composite topping	—	2.30

Evaluation of long-term camber and deflection predictions based on MDOT and PCI multipliers as well as proposed prediction methodology is provided in section 8.3.

2.5.2. Incremental Time Step Method (Nilson 1987)

The Incremental Time Step Method (ITSM) (Nilson 1987) is based on the idea that correctly capturing the variation of the prestressing force with time leads to a correct estimation of time-dependent camber and displacement. The design life of the structure is divided into several increasingly larger time intervals (ACI 435R-95). PCI recommends that at least four time steps are used when implementing this method and requires the adoption of creep, shrinkage, and relaxation time-dependent functions. The strain distributions, curvatures, and prestressing forces are calculated for each interval together with the incremental shrinkage, creep, and relaxation losses

during the particular time interval. (ACI 435R-95). The procedure is repeated for all subsequent incremental intervals, and an integration or summation of the incremental curvatures is made to give the total time-dependent curvature at the particular section along the span (ACI 435R-95). These calculations should be made for a sufficient number of points along the span to allow the construction of a curvature diagram with reasonable accuracy. This diagram can then be used to calculate deflections using numerical integration and the second moment area theorem as explained above. Eqn. 2-20 can be used to determine curvature at a given section in a prestressed concrete beam at any given point in time.

$$\phi_{pt} = -\frac{P_i e_x}{E_{ci} I_c} + \sum_0^t (P_{n-1} - P_n) \frac{e_x}{E_c I_c} - \sum_0^t (C_n - C_{n-1}) P_{n-1} \frac{e_x}{E_c I_c} \quad 2-20$$

Where:

ϕ_{pt} is the curvature at any section and at any time t caused by prestressing effect

P_i is the prestressing force at release

e_x is the eccentricity of prestressing strands with respect to beam centroid at section x

E_{ci} is the modulus of elasticity of concrete at release

E_c is the modulus of elasticity of concrete at considered time interval

I_c is the moment of inertia of beam

P_{n-1} is the prestressing force at the beginning of time interval

P_n is the prestressing force at the end of time interval

C_{n-1} is the creep coefficient at the beginning of time interval

C_n is the creep coefficient at the end of time interval

The first term in Eqn. 2-20 gives the immediate curvature due to prestressing effect. The second term is a reduction due to prestress loss. The third term is increase in curvature due to prestress force induced creep effect. As can be seen, a creep model needs to be adopted to implement this method and a number of time steps. It is implied in this method that the user is going to adopt a methodology to estimate prestress losses at every considered time step. Detailed guidance for how to estimate prestress losses is provided in ACI 423.10R-16. The method also allows for the use of various moduli of elasticity at every time step considered provided that a time-dependent function for modulus is employed. One possible approach to implement the Incremental Time Step Method (Nilson 1987) considering four time steps is as follows:

Step 1 (t_0): Prestress Transfer – Determine all immediate losses. Determine force in the tendon immediately after transfer. $P_1 = P_j - (\text{Elastic Shortening (ES)} + \text{Relaxation (RE)} + \text{Anchorage Seating})$.

Step 2 ($t_0 - t_1$): Prestress Transfer to 30 Days – Calculate losses during this period using P_1 and calculate $P_2 = P_1 - CR_{1-2} - RE_{1-2} - SH_{1-2}$ where CR and SH stands for creep and shrinkage. This can be done as follows:

- Calculate strain in concrete at the centroid of tendons, ε_0 .
- Calculate creep coefficient $\phi(t_1, t_0)$.
- Calculate creep strain $\varepsilon_0\phi(t_1, t_0)$.
- Calculate shrinkage strain $\varepsilon_{SH}(t_1, t_0)$.
- Calculate creep and shrinkage prestress loss $E_{ps}[\varepsilon_0\phi(t_1, t_0) + \varepsilon_{SH}(t_1, t_0)]$, where E_{ps} is the modulus of elasticity of strands.
- Calculate relaxation loss (select a function).
- Calculate $P_2 = P_1 - CR_{1-2} - RE_{1-2} - SH_{1-2}$.

Step 3 ($t_1 - t_2$): 30 days to 1 year – Calculate losses during this period of time using P_2 and calculate $P_3 = P_2 - CR_{2-3} - RE_{2-3} - SH_{2-3}$.

- ❖ First, considering the loads applied at t_0 (P_1):
 - Calculate creep coefficient $\phi(t_2, t_0)$ and subtract $\phi(t_1, t_0)$.
 - Calculate creep strain for loads applied at t_0 , $\varepsilon_0[\phi(t_2, t_0) - \phi(t_1, t_0)]$ (Figure 2.10).
- ❖ Second, considering the loads applied at t_1 (P_2):
 - Calculate strain at the centroid of tendons due to loads applied at t_1 , ε_1 .
 - Calculate creep coefficient $\phi(t_2, t_1)$
 - Calculate creep strain $\varepsilon_1\phi(t_2, t_1)$
 - Calculate shrinkage strain $\varepsilon_{SH}(t_2, t_1)$
- ❖ Third,
 - Calculate creep and shrinkage prestress loss over interval: $E_{ps}[\varepsilon_0[\phi(t_2, t_0) - \phi(t_1, t_0)] - \varepsilon_1\phi(t_2, t_1) + \varepsilon_{SH}(t_2, t_1)]$.
 - Calculate relaxation loss over interval.
 - Calculate force in tendon at the end of time step $P_3 = P_2 - CR_{2-3} - RE_{2-3} - SH_{2-3}$.

Step 4: 1 year to end of service life – Repeat.

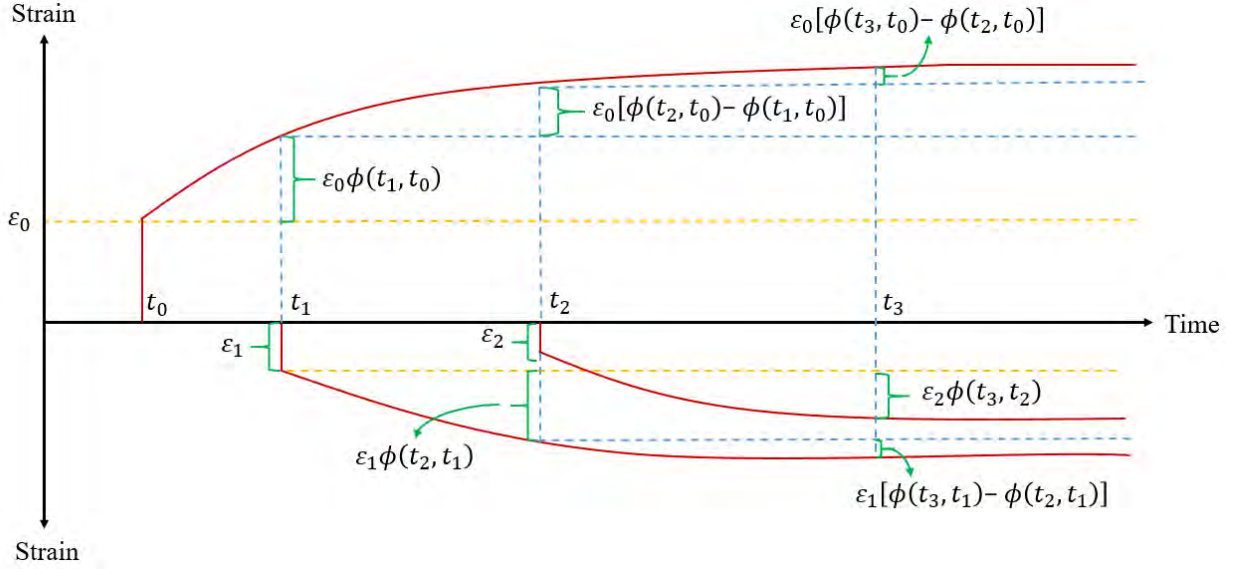


Figure 2.10. Illustration of creep induced strain due to prestress and prestress loss

Once the prestress losses for every time step are calculated, they can be implemented in Eqn. 2-20 to calculate curvature at various sections along the beam and then camber using the second moment area theorem (section 2.4.1).

Pre-erection camber predictions based on the Incremental Time Step Method (Nilson 1987) are compared to predictions based on the proposed methodology in section 8.5. It should be noted that the Incremental Time Step Method (Nilson 1987) is intended to only capture the variation in the prestressing force with time and how that force affects camber and deflection. However, this method cannot capture flexural deformations due to phenomena like differential shrinkage between the deck and the girder or shrinkage induced creep. Therefore, the comparison with the proposed methodology is conducted only for pre-erection camber and not after the deck is cast, because the downward displacement caused by differential shrinkage is significant and cannot be captured by the Incremental Time Step Method (Nilson 1987).

2.5.3. Axial Strain and Curvature Method (Ghali and Favre 1986)

This method originally developed by Ghali and Favre (1986) and further discussed in Ghali (1986) and Elbadry and Ghali (1989) is based on a strain compatibility based cross-sectional analysis, which is used to calculate curvatures and deflections. It provides a procedure to estimate both instantaneous and long-term deflections of members by considering cracked section properties. The change in deflection is calculated by using the slope of change in strain diagram, which gives the change in curvature. Besides, for prestressed members, it is not required to calculate the prestress losses. However, the modulus of elasticity of concrete is age-adjusted by introducing an ageing coefficient (Trost 1967, Bazant 1972). Moreover, using age-adjusted effective modulus (AAEM), transformed section properties are utilized to determine the change in forces and strains. The AAEM expression is given by Naaman (2004) as follows:

$$E_{AAEM}(t, t_0) = \frac{E_c(t, t_0)}{1 + \chi C_r} \quad 2-21$$

Where:

E_{AAEM} is the age-adjusted effective modulus,

E_c is the modulus of elasticity of concrete,

χ is the ageing coefficient, may be taken as 0.7 for practical purposes,

C_r is the creep coefficient.

Using compatibility and equilibrium equations throughout the analysis together with the adjusted material properties mentioned above, deflection is determined by evaluating the curvature at various section along the span. This method is explained in detail in Ghali et al. (2002).

2.5.4. Approximate Time Step Method (Branson and Ozell 1961)

The approximate time-step method, also known as the creep coefficient and average prestress force method (Nilson 1987), is a simplified version of the incremental time steps method (Nilson 1987) and it includes an initial and a final time step. It is originally presented by Branson and Ozell (1961), and ACI 435R-95. It is quite similar to multiplier method such that it provides a procedure for estimation of long-term camber such that each effect such as prestressing, self-weight of girder, and superimposed dead load is considered separately. Then, each of them is amplified by ultimate creep coefficient. Specifically, for prestress effect, a procedure in prestress loss method (Tadros et al. 1985) is proposed such that average prestressing force is used for long-term deflection and a reduction is applied due to prestressing loss. After their contributions are determined to long-term deflection, they are simply summed up and final net deflection is obtained.

$$\delta_T = -\left(\frac{\delta_i + \delta_e}{2}\right) C_u + (\delta_G + \delta_{SD})(1 + C_u) + \delta_L \quad 2-22$$

Where:

δ_T is the total net deflection,

δ_i is the deflection caused by initial prestressing force,

δ_e is the deflection caused by effective (long-term) prestressing force,

δ_G is the deflection caused by girder self-weight,

δ_{SD} is the deflection caused by superimposed dead load,

δ_L is the deflection caused by live load,

C_u is the ultimate creep coefficient.

As seen in Eqn. 2-22, the terms for live load deflection is added for the computation of the long-term deflection, however it should be noted that it is not magnified by creep coefficient because it just causes instantaneous increase or decrease in deflection of the member. Moreover, according to ACI 435R-95 (1995), this method tends to yield, in most cases, comparable results to PCI multiplier method.

2.5.5. Prestress Loss Method (Tadros et al. 1985)

This method is outlined in Tadros et al. (1985) and uses a set of time-dependent multipliers for predicting camber and displacements at discrete points in time similar to the method proposed by Martin (1977). However, unlike Martin (1977), this method provides a time-dependent coefficient for prestress losses due to creep, shrinkage, and relaxation. It should be noted that the time-dependent multipliers in question are provided at discrete times namely at erection, final time, and long term, which is defined as the difference between final time and erection time.

2.5.6. CEB-FIB Model Code (CEB-FIP 1990)

This method provides detailed and simplified approaches for evaluating deflection and camber in prestressed concrete elements, cracked or uncracked (ACI 435R-95). Details of this approach are given in CEB-FIB (1990). The simplified method includes the use of multipliers that account for long terms effects including creep, cracking, and percentage of reinforcement.

2.5.7. Section Curvature Method (ACI 435R-95)

This method is provided in Appendix B of ACI 435R-95 and is a comprehensive method for the computation of deformations in plane frames. The method is based on a strain compatibility based cross-sectional analysis, which is used to obtain axial strains and curvatures along the length of the members with the purpose of calculating frame deflections, translations and rotations. The

method requires the use of creep and shrinkage models to account for time dependent effects and employs the AAEM method (Trost 1967, Bazant 1972).

2.6. Factors that Affect Camber at Release (Initial Camber) and Long-term Camber, Deflection and Rebound

One of the reasons why camber, deflections, and rebound are difficult to predict accurately is the uncertainty in quantifying the various factors that affect beam behavior. Consider, for instance, the simple case of a simply supported PC beam with a harped tendon profile loaded with self-weight only. Deflection at midspan due to self-weight is given by Eqn. 2-6, while camber due to prestressing force is given by Eqn. 2-10.

Although both are theoretically ‘exact’ models, none of the input parameters are usually precisely known. For example, the modulus of elasticity at the time of prestress release, a key factor for determining camber at release and long-term camber, deflections, and prestress losses, is typically calculated using approximate formulas as a function of compressive strength, that are known to have a large amount of scatter (Pauw 1960; Raphael 1985). Although perhaps known with less uncertainty, beam self-weight, the initial prestressing force, beam length, location of harping points, cross-sectional geometry, and prestress eccentricities at the ends and mid-span, are all estimates of actual conditions. Other factors that affect camber at release are unintended cracking, inaccurate estimates of transfer length, and any debonding between the strands and concrete. Storm et al. (2013) investigated the effects of fabrication practices on prestressed concrete beams and concluded that the actual compressive strength at release, the deformation of internal void forms in box beams during casting, and curing method all significantly affect camber.

The effect of these uncertainties is further compounded when long-term behavior accounting for creep and shrinkage is considered. When a beam is cast using typical fabrication methods and placed within an actual structure, a multitude of factors may significantly affect the

magnitude of beam camber and deflections. For example, downward deflections are not only caused by beam and deck self-weight and associated creep, but also by other factors such as: differential shrinkage between the beam and strands in the initial non-composite section; differential shrinkage between the deck and beam in the final composite section; and negative temperature gradients. Conversely, camber is primarily caused by the prestressing force and associated creep as well as positive temperature gradients. In both cases, concrete creep and shrinkage will vary depending on the initial concrete properties; the member size effect (volume to surface ratio); the method and length of curing; age at loading; level and variation of prestressing force and other sustained loads; as well as environmental conditions.

Additional factors will affect the rebound of all beam types, such as the weight of the removed deck, unintended support stiffness, connectivity to other components such as diaphragms and end walls, as well as member deterioration.

2.7. Prestressed Concrete Beam Questionnaire

A prestressed concrete beam camber questionnaire consisting of 31 questions was distributed nationwide with the purpose of gaining a better understanding of the current nationwide practice for estimating the camber and deflection of bridge superstructures. The questions that were included in the survey as well as the answers received are provided in APPENDIX B.

Almost 70% of the 40 respondents reported that they have experienced camber related problems with prestressed concrete bridge beams that are often beyond normal construction tolerances. Among the responses related to camber related problems given by the engineers, the fact that the one reason behind inaccurate camber estimation is keeping the beam in the precast yard more than it is planned was the most repetitive one.

As stated at the beginning of this chapter and as corroborated by the results of the survey, at least 13 states have sponsored camber related research with the purpose of improving methods for predicting camber.

When asked what method the respondents use to predict prestressed concrete beam camber; 45.95% of the 37 respondents indicated that they used the multiplier method; 27.03% indicated that they used the prestress loss method; 2.70% indicated that they used the approximate time step method; another 2.70% indicated that they used the incremental time step method; and 21.62% indicated that they use other methods. These results suggest that the number of states that use the theoretically most accurate incremental time step method, a version of which is used in this study, is rather small (1 out of 37 respondents).

When asked whether the method that the respondents use considers explicitly the effects of creep and shrinkage only 9 out of 43 respondents provided answers. Out of these 9 respondents 56% responded “yes” and 44% responded “no”.

When asked which model the respondents used to predict the effects of creep and shrinkage, 25 out of 32 respondents (78%) indicated that they used the AASHTO LRFD Specification method, 1 respondent indicated that they used the ACI 209R-92 model, and 6 respondents indicated that they used other methods. However, out of the six respondents who chose other methods, one referred to the recommendations in the NCHRP Report 496, one mentioned the name of the software used to calculate camber, one indicated that only creep effects are accounted for, and rest indicated that creep and shrinkage are not explicitly accounted for.

When asked whether the respondents had a tool that they used to predict camber and deflection at a given time after the erection of the deck and after the opening of the bridge to traffic, the majority of the respondents responded “no”. A few referenced the “Leap Concrete Bridge

Software by Bentley”, some referenced the multiplier method, some responded that camber and deflection after the deck is cast is not of interest, and some referenced in-house programs and procedures.

When asked what the range of concrete compressive strength at release and 28 days that the respondents typically specified was, the response most of the time included a range between 4-8 ksi. Lower and upper bounds included 4 ksi and 12 ksi.

When asked how the respondents determined the modulus of elasticity in camber predictions, 24 out of the 30 respondents (80%) indicated that they used the equation in the AASHTO LRFD Specifications, and 9 out of 30 indicated that they used other methods. However, many of the 9 respondents indicated that they used a version of the AASHTO LRFD Specification equation with various values for coefficient K . For example, one respondent indicated that use of $K = 0.9$ because of old Nebraska aggregate. Other respondents indicated they use $K = 0.85$, and another indicated that they use $K = 1.0$. As can be seen some type of regional calibration is provided to account for local conditions. TxDOT policy is to set all concrete moduli to 5000 ksi for stress and deflection calculations. One respondent indicated that they used the equation in ACI 363, and another indicated that they had derived their own equation based on curve fitting techniques using historical data and provided the equation.

When asked whether the respondents used the specified value for compressive strength or some other value when calculating modulus using the equation provided in AASHTO LRFD Specifications, most respondents stated that they used specified values. One respondent stated that the specified release strength is increased by 25% and specified 28-day strength is increased by 45%. Another responded that they have adopted a procedure to adjust based on expected strengths

since they are typically higher. One respondent stated that specified rather than expected values are used since the variability in compressive strength is believed to be included in the multipliers.

When asked whether a specific curing technique was required, 17 out of 32 responded indicated “yes” and the rest indicated “no”. Out of the 17 respondents who indicated that a specific curing technique was required, four indicated that they specified steam curing, 3 moist curing, and the rest chose “other”. Other techniques identified included radiant heat and steam and moist curing depending on the season during fabrication. When asked which curing technique was most prevalent in the state, the answers primarily included steam or moist curing.

When asked which method the respondents used to measure camber, most answers included either the string line or surveying methods. Similar results were reported when asked which tools the respondents used when measuring camber.

When asked how frequently camber was measured, answers included once, twice, three times, weekly, monthly, rarely, and as needed. The policies for when camber was measured varied.

When asked what time of the day camber was measured, most responded that there was not a specific time required and any time during daylight hours was acceptable.

When asked what procedure or approach (such as pre-loading) was used to minimize or control camber overgrowth most responded that either they did not resort to such techniques or used either rarely or occasionally.

When asked how inaccurate camber predictions were typically dealt with, respondents reported various techniques such as adjustments in haunch height, seat elevations, screed elevations, stirrup extensions, and asphalt overlay.

When asked whether box beams were prone to inaccurate camber predictions, 13 out of 27 respondents stated “yes”, 6 “no”, and 8 indicated that this beam type was not used in their state. It is interesting to note how the box beams are not used in 8 states. In general, no specific box beam size was indicated to be more prone to inaccurate camber predictions.

When asked whether cored slabs were prone to inaccurate camber predictions, 4 out of 28 respondents stated “yes”, 5 “no”, and 19 indicated that this beam type was not used in their state.

When asked whether AASHTO I-beams were prone to inaccurate camber predictions, 7 out of 30 respondents stated “yes”, 11 “no”, and 12 indicated that this beam type is not used in their state.

When asked whether bulb T beams were more prone to inaccurate camber predictions, 14 out of 29 respondents stated “yes”, 11 “no”, and 4 stated that this beam type was not used in their state.

When asked whether other beam types were prone to inaccurate camber predictions, 7 out of 30 respondents indicated “yes”, 15 “no”, and interestingly 8 respondents indicated that this beam type was not used in their state.

When asked whether the respondent had a procedure or tool to predict beam rebound in deck replacement projects, most respondents indicated “no”.

When asked whether the respondents had measured beam rebound in existing beams after deck weight is removed and the deflection of the beams after the new deck has been placed, 10 out of 28 respondents indicated “yes”, and 18 “no”. When asked whether a correlation existed between rebound and deflection, the answers among the 10 respondents varied between “no” and “yes”.

In summary, the results of the national survey were useful in revealing the practices of various states related to camber prediction and provided context for the various tasks discussed as part of this thesis.



CHAPTER 3: Prediction Methodology

3.1. Introduction

In this chapter, the proposed prediction methodology, named as “Time-Dependent Cross-sectional Curvature Method, for estimating short-term and long-term camber and deflections is discussed. In sections 2.4 and 2.5, available methods for estimating short-term and long-term camber and deflections are explained in detail, respectively. As stated before, in this thesis, moment-area theorem is utilized to compute the camber and deflections. In section 2.4.1, explanations about how the moment-area theorem is implemented are provided comprehensively. In fact, some formulas are derived to estimate the short-term camber and deflections of prestressed concrete beams for specific prestressing strand profiles.

Furthermore, the assumptions made in proposed prediction methodology and adopted sign convention are provided. Since the prediction of long-term camber and deflection involves many time-dependent parameters such as creep, shrinkage, relaxation and modulus of elasticity, it is important to have accurate models for such parameters to capture the time-dependent effects accurately. For this, various models for such parameters are provided and their differences from each other are highlighted. Finally, explanations about temperature gradients based on AASHTO LRFD Bridge Design Specifications (2020) are presented, and a brief introduction for proposed Time-Dependent Multiplier method is provided.

3.2. Time-Dependent Cross-sectional Curvature Method

In this part of the thesis, the proposed prediction methodology is discussed and the approach for its development is presented. The proposed prediction methodology is general and is able to predict camber and displacements at any point during the life of a prestressed concrete beam or composite bridge superstructure featuring prestressed concrete or steel beams and a

concrete deck. The overall framework for the prediction methodology is based on principles of engineering mechanics, although components of the methodology are based on empirical models, such as the estimation of the modulus of elasticity for beam and deck concrete, relaxation of steel strands, and prediction of concrete creep and shrinkage properties. The proposed prediction methodology for short-term and long-term camber was developed based on several iterations/approaches as shown in Table 3.1. While two iterations are presented for short-term camber i.e. camber at release, six methods are provided for long-term camber, i.e. service camber. Approach No. 6 represents the final version of the proposed prediction methodology. The details of the other approaches are presented with the purpose of discussing their advantages and limitations and to provide some context for comparisons presented later in this report between camber predictions obtained using various approaches. The implementation of Approach No. 6 is presented in APPENDIX D and APPENDIX G through the use of a Matlab based algorithm that features a graphical user interface. Examples of how the algorithm used in Approach No. 6 works are provided in APPENDIX E and APPENDIX F for composite prestressed concrete and steel beams, respectively.

All approaches are based on the principle of superposition. The principle of superposition was first applied by McHenry (1943) who stated that the strain produced by a stress increment applied at any time τ_i is not affected by any stress applied either earlier or later. Gilbert and Ranzi (2010) report that for increasing stress histories, the principle of superposition agrees well with experimental observations, whereas for decreasing stress histories it overestimates creep recovery. In the case of composite systems, increasing stress histories include the placement of deck and superimposed dead loads, whereas decreasing stress histories include time-dependent prestress losses and deck removal. In general, it has been reported (Gilbert and Ranzi 2010) that the principle

Table 3.1. Description of approaches used to develop the proposed prediction methodology

A: Ignore debonding, transfer length, and variation in support location, use gross section properties.
 B: Consider debonding, transfer length, and variation in support location, use transformed section properties.
 C: Number of sections = end and midspan
 D: Number of sections = Multiple
 E: Creep effects = AAEM, $\mu=0.7/1.0$, single creep curve, discrete time steps, number of sections = end and midspan
 F: Creep effects = EM, multiple time steps, single creep curve, Mod. = $f(t)$, number of sections = end and midspan
 G: Creep effects = EM, multiple time steps, single creep curve, Mod. = $f(t)$, number of sections = multiple
 H: Creep effects = EM, multiple time steps, multiple creep curves, Mod. = $f(t)$, number of sections = multiple
 NA: Not applicable

3.3. *Primary Assumptions and Sign Convention*

Throughout the derivations of proposed methodology and analyses conducted in this thesis, the following assumptions are made:

- plane cross-sections before bending remain plane after bending. By this way, a relationship between change in strains at different layers along the cross-section and curvatures can be obtained.
- Member is fully prestressed i.e. there is no cracking on the member. With this assumption, throughout the calculations, gross or transformed moment of inertia values are used, and predictions are performed accordingly.
- There is a perfect bond between strands and concrete. It permits to use strain compatibility equations.
- The cross-sections have one axis of symmetry and subject to bending moment and axial forces caused by prestressing or other loading.
- Horizontal shear transfer between deck and beam is perfect. In other words, there is no slippage i.e. relative movement between girder and deck. It enables using strain compatibility while capturing the differential shrinkage effect between deck and girder.
- Members behave fully linearly-elastic. This assumption enables to use Hooke's law, which states that strain is linearly dependent on stress.
- Shear deformations are neglected. It assumed that they are negligible when compared to flexural deflections.
- Beams are symmetric about their midspan. This provides symmetric curvature diagram about midspan and calculations are performed accordingly.

- Girder with or without deck is simply supported such that the elongations and end rotations are not restricted. This makes the structure statically determinate and facilitates the structural analysis calculations.
- Unshored construction technique is employed when the deck and other superimposed dead loads are applied. That is the deck load is carried by not the composite section but girder section alone.
- Girder, deck, and additional superimposed dead loads are uniformly distributed along the span length of girder. Only the diaphragm load is considered as a point load up to 3 diaphragms. After that number, it is also considered as uniformly distributed load.
- While considering self-weight effects in the analysis, unit weight of concrete is increased by 5 pcf to include the weights of prestressing strands and mild reinforcement, as recommended in AASHTO LRFD Bridge Design Specifications (2020). (This was assumed only for the analyses. User can enter any value for the unit-weight of concrete).
- No reduction is applied for barrier load. In other words, entire barrier load is carried by one girder.
- Initial deck concrete compressive strength is equal to 80% of its 28 day strength. Since initial deck concrete compressive strength generally is not specified, this assumption is made, as suggested AASHTO LRFD Bridge Design Specifications (2020).
- It is assumed that deck starts carrying load based on its stiffness 1 day after the it is cast.
- Reduction in surface area of girder when deck is placed is ignored. It enables using one function for creep and shrinkage.
- Transfer length for prestressed concrete beams equals to $60d_{ps}$ where d_{ps} is the strand diameter, as suggested by AASHTO LRFD Bridge Design Specifications (2020).

- Mean concrete compressive strength is 1.2 ksi more than specified value, as suggested by ACI 209.2R-08.
- For the variation of modulus of elasticity of deck concrete with respect to time, ACI 209R-92 model with Type I cement and moist curing is selected.
- For deck concrete modulus of elasticity, AASHTO LRFD Bridge Design Specifications (2020) model is used.
- Relaxation of prestressing strands is neglected between time at tensioning and time at transfer.
- Transformed section properties are utilized.
- Deflections are calculated according to the girder end, not the bearing location.
- Tension creep is assumed to be equal to compression creep. This provides same creep models with compression and tension forces. (As mentioned above, magnification factors for tensile and flexural creep are provided. They are assumed to be 1.0 in the analysis, but user can enter any value for these).

The sign convention utilized in this thesis is shown in Figure 3.1. All the formulations, derivations, analysis results and discussions are based on this unless otherwise stated. Upward deflections caused by prestressing or other loadings are positive. Axial force is positive when tensile, and bending moment that generates tension at the bottom fiber and corresponding curvature are positive.

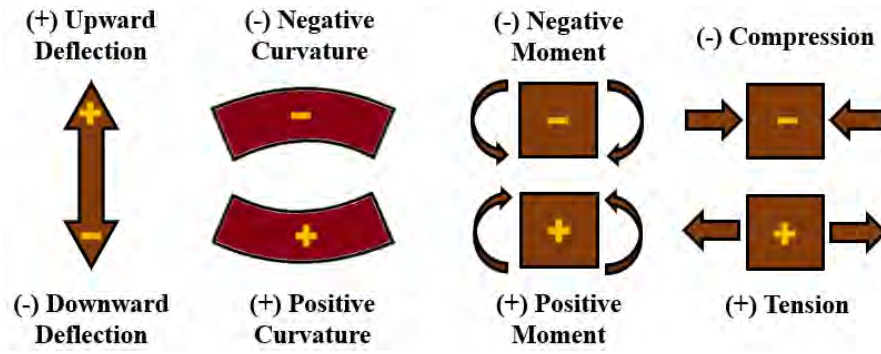


Figure 3.1. Adopted sign convention

3.4. Prediction Methodology for Camber at Release

Since most of the steel girders are intentionally cambered during fabrication process to compensate the downward deflections caused by deck and other superimposed dead loads as well as differential shrinkage between steel and deck, this section does not apply for them. Therefore, this part covers the camber at release for the prestressed concrete girders only.

Typically, estimation of camber at release for prestressed concrete beams is composed of two main parts: downward deflection caused by beam self-weight and upward deflection, i.e. camber caused by prestressing effect. The latter may be difficult to calculate when compared to the former one because there are more parameters involved in calculation of camber due to prestressing effect such as transfer length, existence of debonded and harped strand. The formulations given in section 2.4.1 for estimating short-term deflection do not consider all of these parameters. However, to be able to obtain an accurate prediction of camber and displacements, these parameters are needed to be taken into account.

The proposed prediction methodology for camber at release developed based on two iterations/approaches is shown in Table 3.1. The differences between them as well as the difference between basic and advanced approaches are discussed in the sections 3.4.1 and 3.4.2 below.

3.4.1. Approaches No. 1 to 4

In Approach No. 1 in Table 3.1, two levels of analyses were featured: 1) basic and 2) advanced. Both levels of analyses were used to predict camber at release. In the basic approach, strand debonding, transfer length, and variation of support location were not considered. In addition, if there are harped strands, they were assumed to be harped at $0.4L$ from girder ends where L is the overall length of the beam, and the support locations were assumed to be at the ends of the beam. Elastic shortening loss was calculated using gross cross-sectional section properties and the closed form equation provided in the commentary of AASHTO LRFD Bridge Design Specifications (2020) as given in 3-122. While calculating the camber at release, gross cross-sectional properties with effective prestressing force immediately after transfer were used. On the other hand, in the advanced approach, the location of supports, transfer length, debonding, and multiple strand patterns were all taken into account. Moreover, unlike basic approach, transformed cross-sectional properties were utilized in this approach. Elastic shortening loss was not deduced from jacking prestressing force when calculating camber at release because using transformed section properties inherently takes the effect of elastic shortening into account. This resulted in similar estimations for camber at release with those obtained using the basic approach.

In both basic and advanced approach, the effects of girder self-weight and prestressing were considered separately, and only midspan and end sections were considered in the calculations. In other words, the girder section properties were calculated once for the girder end and once for the midspan. Depending on the existence of harped strand configuration, these properties may differ from each other. Moreover, camber was calculated as the difference in beam elevation at mid-span with respect to the supports. The supports can be temporary supports during storage at the precast facility. Camber predictions were conducted in Mathcad.

For the basic approach, Eqns. 2-6 and 2-10 were utilized by assuming the coefficient α equals to 0.40 all the time. It should be noted that while using Eqn. 2-10, moment of inertia at girder midspan and ends were utilized (Figure 2.7). On the other hand, formulations for advanced approach in this approach (i.e. Approach No. 1), and Approaches No. 2, 3 and 4 are provided below.

The self-weight effect on deflection is calculated as follows:

$$\Delta_{Gsw} = A_1 \bar{x}_1 + A_2 \bar{x}_2 \quad 3-1$$

$$A_1 = \frac{2}{3} \left(\frac{L_G - 2L_t}{2} \right) \left[\frac{w(L_G - 2L_t)^2}{8EI_m} \right] \quad 3-2$$

$$\bar{x}_1 = \frac{5}{8} \left(\frac{L_G - 2L_t}{2} \right) \quad 3-3$$

$$A_2 = \left(\frac{L_G - 2L_t}{2} \right) \left(-\frac{wL_t^2}{2EI_e} \right) \quad 3-4$$

$$\bar{x}_2 = \left(\frac{L_G - 2L_t}{2} \right) \quad 3-5$$

Where:

Δ_{Gsw} is the effect of self-weight on deflection,

A_1 and A_2 are the areas under curvature diagrams, as shown in Figure 3.2,

$\bar{x}_1$ and $\bar{x}_2$ are the centroids of the areas A_1 and A_2 , as shown in Figure 3.2,

L_G is the girder length from end to end,

L_t is the girder overhang length,

w is the applied load

E is the modulus of elasticity,

I_e and I_m are the moment of inertia values at the end and midspan, respectively.

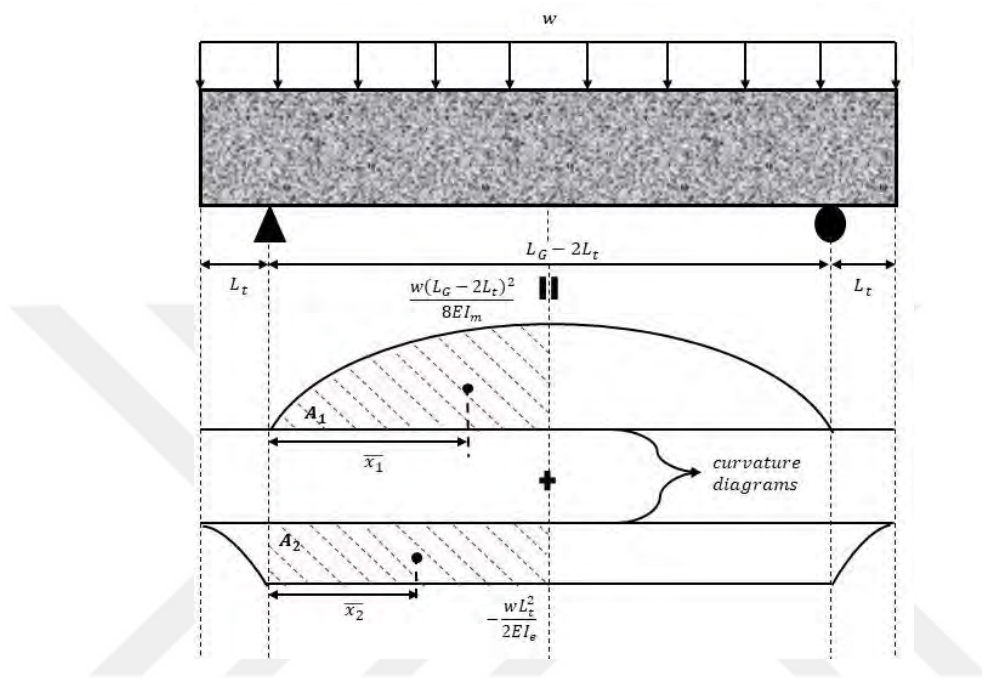


Figure 3.2. Curvature diagrams for self-weight effect only

It can be realized that for the second curvature diagram shown in Figure 3.2, although midspan and end curvature values caused by overhang moments may not be identical, it is assumed to be same to facilitate the computations. Since the difference between I_m and I_e is not significant and the overhang moment is so small compared to midspan moment, this approximation does not lead to significant error in the calculations. This assumption will be removed together with the Approach No. 5 where multiple sections along the span are used.

Similarly, the effect of prestressing on deflection can be calculated by using the moment-area theorem in Figure 3.3. However, in this case, calculations are a bit complex when compared to self-weight' because depending on the location of bearing supports, the formulation may change.

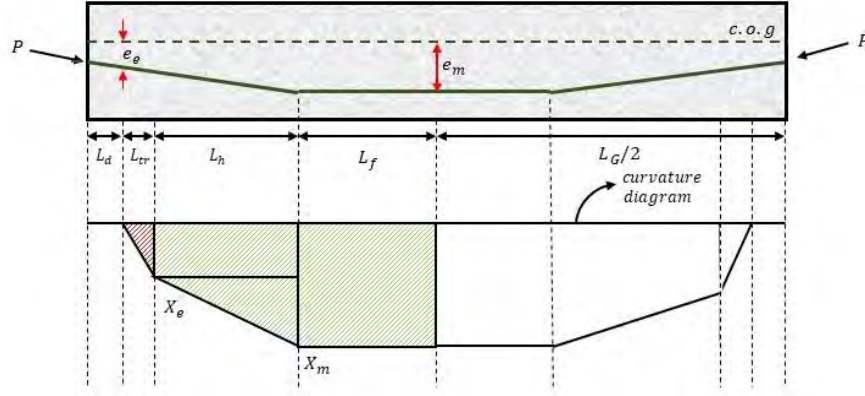


Figure 3.3. Curvature diagram for prestressing effect only

To facilitate the computations, it is assumed that locations of bearing points from girder end do not exceed the sum of debonding and transfer length. This assumption is always valid for the database used in this thesis (Table 4.1). However, this assumption will be eliminated with the introduction of Approach No. 5 where multiple sections along the span are used and camber is predicted with respect to girder end rather than location of supports.

$$\Delta_{ps} = X_m L_f \left(L_s + L_h + \frac{L_f}{2} \right) + X_e L_h \left(L_s + \frac{L_h}{2} \right) + \frac{X_e L_{tr}}{2} \left(L_d - L_t + \frac{2L_{tr}}{3} \right) \quad 3-6$$

If $L_t \leq L_d$

$$+ \frac{(X_m - X_e)L_h}{2} \left(L_s + \frac{2L_h}{3} \right)$$

$$\Delta_{ps} = X_m L_f \left(L_s + L_h + \frac{L_f}{2} \right) + X_e L_h \left(L_s + \frac{L_h}{2} \right) \quad 3-7$$

If $L_d < L_t \leq (L_{tr} + L_d)$

$$+ \frac{X_e L_s^2}{2} \left(\frac{2L_{tr} + L_t - L_d}{3L_{tr}} \right) + \frac{(X_m - X_e)L_h}{2} \left(L_s + \frac{2L_h}{3} \right)$$

$$L_s = L_d + L_{tr} - L_t \quad 3-8$$

$$L_f = \frac{L_G}{2} - (L_d + L_{tr} + L_h) \quad 3-9$$

Where:

Δ_{ps} is the effect of prestressing on deflection,

X_m and X_e are the midspan and end curvature values, respectively,

L_f the length of section between end of harping point to midspan,

L_s is the parameter about bearing locations,

L_h is the effective harping length,

L_{tr} is the transfer length, assumed to be $0.6d_{ps}$ where d_{ps} is the diameter of prestressing strand,

L_d is the debonded length,

L_t is the location of bearing points measured from girder end,

L_G is the girder length measured from end to end.

Eqns. 3-6 and 3-7 are obtained by taking the first moment of the green and red areas in Figure 3.3 with respect to the location of bearing supports based on the assumption stated above. Although these equations seem to be complex, they take the effect of harping, transfer length, debonding and existence of overhang into account. If there are no harped or debonded strands, the parameters about them are set to zero. By this way, all possible strand configurations are taken into account.

3.4.2. Approaches No. 5 and No. 6

In the Approaches No. 5 and 6, as well as in Approaches No. 2, 3 and 4, the basic level of analysis was removed and only the advanced analysis was further developed since all calculations were conducted in Matlab and, in Mathcad for Approaches No. 2, 3 and 4, and there was no convenience to implement simplifying assumptions.

Unlike in previous approaches, the proposed methodology approaches the problem such that it combines the effect of prestressing and self-weight to calculate the camber at release by using second moment area theorem. The step-by-step procedure for proposed approach is provided below.

- 1- Divide the half of the girder length into sections. For each section, do the following steps:
 - i. Determine the eccentricity of prestressing strands. It should be noted that, the eccentricity may change along the half-span due to the existence of harped and debonded tendons. Therefore, the eccentricity may be section-dependent.
 - ii. Determine the cross-sectional area of prestressing strands. It should be noted that if there is any debonded strand, the cross-sectional area of strands is also section-dependent.
 - iii. For the analysis, using principles of transformation from Mechanics of Materials, transform prestressing steel to the concrete.
 - iv. Calculate the transformed cross-sectional area. It should be realized that the transformed cross-sectional area is also section-dependent if cross-sectional area of strands along the half-span is section-dependent.
 - v. Calculate the centroid of transformed section, and using parallel axis theorem, calculate the transformed moment of inertia.
 - vi. Calculate the moment caused by girder self-weight. The expressions for this are as follows:

$$\text{if } x \leq L_t \quad M_{Gsw}(x) = -\frac{w_G x^2}{2} \quad 3-10$$

$$\text{if } L_t < x \qquad M_{Gsw}(x) = \frac{w_G L_G}{2} (x - L_t) - \frac{w_G x^2}{2} \qquad 3-11$$

Where:

M_{Gsw} is the moment at location x , measured from the end of the girder,

w_G is the uniformly distributed load caused by girder self-weight,

L_t is the girder overhang length,

x is the distance measured from end of the girder.

- vii. By using previously determined eccentricity and cross-sectional area of prestressing strands, determine the moment caused by prestressing strands.
- viii. Determine the net moment acting on the section by simply summing the moments due to self-weight of girder and prestressing effect.
- ix. Calculate the curvature by dividing the net moment on the section by flexural stiffness of the member, EI_{ts} , where E is the modulus of elasticity, and I_{ts} is the transformed moment of inertia which is calculated in step (v).

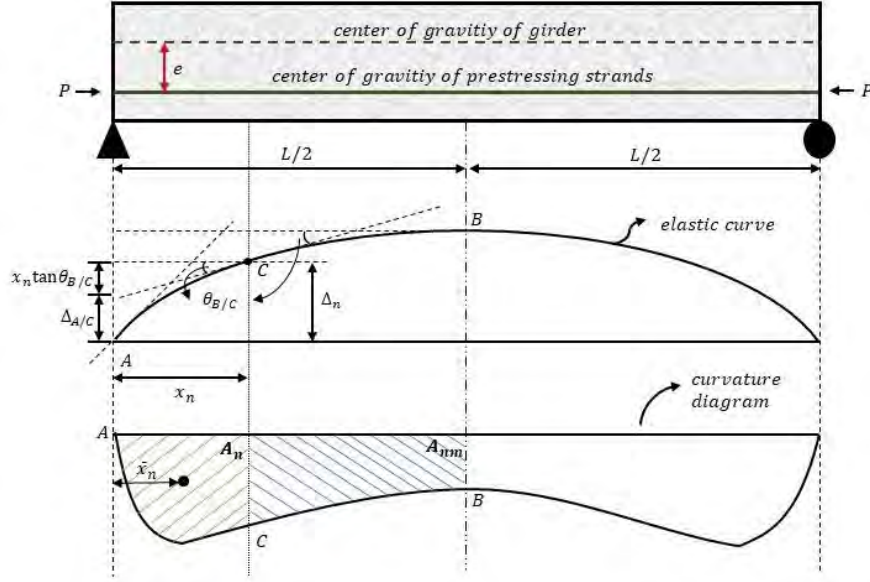


Figure 3.4. Implementation of Moment-area theorem for a typical prestressed concrete beam

- 2) After determining curvatures at each section, use the second moment-area theorem to calculate the net deflection caused by girder self-weight and prestressing effect at each section along the span. This procedure requires a numerical integration such as trapezoidal rule, 1/3 Simpson rule, or Gauss quadratic rule. Even though it is simple to apply, since it provides good accuracy with reasonable number of sections, the 1/3 Simpson rule is preferred in this thesis. To exemplify the usage of this rule with moment-area method to calculate the deflection at any section on the span, consider the simple beam with drawn elastic curve and curvature diagram in Figure 3.4. The net deflection at n^{th} section is calculated as follows:

$$\Delta_n = \Delta_{A/C} + x_n \tan \theta_{B/C} \quad 3-12$$

$$\theta_{B/C} = A_{nm} \quad 3-13$$

$$\Delta_{A/C} = A_n \bar{x}_n \quad 3-14$$

$$\bar{x}_n = \frac{Q_n}{A_n} \quad 3-15$$

$$Q_n = \frac{\Delta x}{3} (\phi_1 x_1 + 4\phi_2 x_2 + 2\phi_3 x_3 + 4\phi_4 x_4 + 2\phi_5 x_5 + \cdots + 4\phi_{n-1} x_{n-1} + \phi_n x_n) \quad 3-16$$

$$A_n = \frac{\Delta x}{3} (\phi_1 + 4\phi_2 + 2\phi_3 + 4\phi_4 + 2\phi_5 + \cdots + 4\phi_{n-1} + \phi_n) \quad 3-17$$

$$A_{nm} = \frac{\Delta x}{3} (\phi_n + 4\phi_{n+1} + 2\phi_{n+2} + \cdots + 4\phi_{m-1} + \phi_m) \quad 3-18$$

$$\Delta x = \frac{L_G/2}{\text{number of sections} - 1} \quad 3-19$$

Where:

Δ_n is the net deflection at nth section according to end of the girder,

$\Delta_{A/C}$ is the relative displacement at point C according to point A,

x_n is the location of nth section measured from closest girder end,

ϕ_n is the curvature at nth section,

x_m is the location of midspan measured from girder end,

ϕ_m is the curvature at midspan,

$\theta_{B/C}$ is the relative slope at point B relative to point C,

A_{nm} is the area under curvature diagram between nth section and midspan,

A_n is the area under curvature diagram between nth section and closest girder end,

Q_n is the first moment of the area under curvature diagram between nth section and closest girder end,

$\bar{x}_n$ is the centroid of the area under curvature diagram between n^{th} section and closest girder end,

Δx is the length of a section,

L_G is the length of girder,

It should be noted that while using the 1/3 Simpson rule, the coefficients for the multiplication of location and curvature should be determined such that the first and the last terms' coefficients are 1; for “odd” number of sections, the multiplier is 4 while “even” number of sections, the multiplier should be 2, as shown in Eqn. 3-16. It can be realized that while calculating the deflection for midspan only, the second term in Eqn. 3-12 comes out to be zero, thus it is enough to use Eqn. 3-16, where Q , first moment of the area, gives the midspan camber directly. Moreover, it should be noted that since this solution is an approximation, the more sections considered means the more accurate results. A discussion about how many sections is required to obtain reasonably accurate results is mentioned in section 8.6.6.

The procedure above would be a quite time-consuming process if it is intended to be employed by hand, especially, if the number of sections investigated is numerous. In the appendix sections E.3 and F.3, step-by-step implementation of this process is shown mathematically. From these sections, it can be seen how tedious to implement this method by hand although the number of sections investigated was only three. Therefore, it is highly suggested to use a programming tool such as Excel, Mathcad and Matlab while employing this method.

In section 8.6.1, results obtained from various approaches i.e. Approach No. 1 to No. 6 are compared until pre-erection time. It should be noted that although all approaches take the bearing locations into account, the Approaches No. 1 to No. 4 predict the deflection relative to support, i.e. bearing locations, whereas Approaches No. 5 and No. 6 predict the camber with respect to girder

ends. Therefore, while comparing camber and deflection results using various approaches, girder overhang lengths are set to zero, meaning that the girders are supported at the far ends, to conduct a fair comparison.

3.5. Prediction Methodology for Long-term Deflection

The proposed prediction methodology for long-term camber and defections, named as the Time-dependent Cross-sectional Curvature Method, are based on engineering mechanics, but utilizes some empirical models to conduct time-dependent analysis. It can be applied for non-composite and composite structures with cast-in-place concrete decks.

The prediction methodology for long-term camber and deflection was developed based on six approaches as presented in Table 3.1. Approach No. 6 is the final and the most advanced version of the proposed prediction methodology. Other approaches are also mentioned in this part of the thesis to highlight the differences and improvements among them and to provide some context for the comparisons using various approaches provided later in thesis (section 8.6).

3.5.1. Approach No. 1

This approach is concerned with the estimation of long-term camber and deflections up to pre-erection time, and is similar to the axial strain and curvature method proposed by Ghali and Favre (1986) mentioned in section 2.5.3 and section curvature method provided in ACI 435R-95.

As in short-term predictions, this approach featured two levels of analyses: 1) basic and 2) advanced. The differences between these two levels were highlighted in detail in section 3.4.1. In this approach, calculations for the pre-erection camber prediction are presented. While doing so, the Age Adjusted Effective Modulus (AAEM) method (Trost 1967; Bazant 1972) is used to account for creep and concrete ageing effects. Pre-erection camber can be calculated at any time before the deck is placed on the beam. Initial stress conditions are calculated when the strands are

detensioned. Only one time step is considered between camber at release and pre-erection camber at the desired time. Cross-sectional analysis based on strain compatibility is conducted to estimate curvatures at midspan and end of the beam, to construct curvature diagrams, and to estimate the camber using second moment area theorem. Camber predictions are conducted in Mathcad and the user has access to the entire code.

In this approach, camber is calculated as the difference in beam elevation at mid-span with respect to the supports. The supports can be temporary supports during storage at the precast facility or permanent supports at the bridge site. It should be noted that the assumption stated above related to support locations is also valid in this approach. Prestress losses due to creep and shrinkage are considered using a time-dependent sectional analysis and are calculated at pre-erection. Similarly, prestresses losses due to the relaxation of steel are calculated at the pre-erection using the formula provided in the NCHRP Report 496, Eqn. 3-119, which provides a time-dependent function for the calculation of relaxation losses. In this formula one of the required inputs is the elastic shortening loss, which is calculated using the gross cross-sectional properties and the closed form formula provided in the commentary of AASHTO LRFD Bridge Design Specifications (2020) (Eqn. C5.9.3.2.3a-1) which is mentioned in section 3.9. Elastic shortening losses are not included when calculating cross-sectional curvatures and beam displacements since transformed section properties were utilized. All calculated long-term losses are used to adjust the magnitude of the prestressing force at the beginning of the subsequent time step so that camber and displacements at service can be calculated accordingly.

In this approach, the time-dependent effects of prestress and self-weight induced creep, and differential shrinkage between the beam and the strands are considered simultaneously, despite the fact that the curvature diagrams for each phenomenon are different as shown in Figure 3.14. This

is done to avoid conducting several strain compatibility based cross-sectional analyses at midspan and beam end for each phenomenon. Pre-erection camber is calculated using a curvature diagram that is most appropriate for the combined effects of these phenomena which is the multi-linear curvature diagram like the one for prestressing effect as shown in Figure 3.3. This approximation is addressed in Approach No. 2 in which the effect of these phenomena is treated separately so that the appropriate curvature diagram can be used in each case.

Figure 3.5 shows the distribution of forces in a prestressed concrete girder. The time-dependent strain at any fiber in the precast girder can be calculated by summing the elastic and creep strains due to initial stresses, elastic and creep strains due to changes in stress, shrinkage strain, and strains due to thermal gradients (Eqn. 3-20).

$$\varepsilon_c(t_n) = \sigma_c(t_0) \frac{1 + \varphi(t_n, t_0)}{E_c(t_0)} + \int_0^{\Delta\sigma_c(t_n)} \frac{1 + \varphi(t_n, \tau)}{E_c(\tau)} d\sigma_c(\tau) + \varepsilon_{sh}(t_n, t_0) + \varepsilon_{TG} \quad 3-20$$

Where:

t_0 and t_n are the ages of concrete when initial stress is applied and when the strain is considered, respectively,

τ is the incremental time between t_n and t_0 ,

$\varepsilon_c(t_n)$ is the total strain at time t_n ,

$\sigma_c(t_0)$ are the stress at time t_0 ,

$d\sigma(\tau)$ is the incremental or decremental stress applied at age τ ,

$E_c(t_0)$ and $E_c(\tau)$ are the modulus of elasticity of concrete at times t_0 and τ , respectively,

$\varphi(t_n, t_0)$ and $\varphi(t_n, \tau)$ are the creep coefficients at time t_n due to load applied at time t_0 and τ , respectively,

$\varepsilon_{sh}(t_n, t_0)$ is the shrinkage strain occurring between the ages t_0 and t_n ,

ε_{TG} is the strain due to temperature gradient.

In Eqn. 3-20, the first term represents the elastic and creep strains due to a stress applied at time t_0 . The second term represents the elastic and creep strains due to changes in stress in the time interval t_0 to t . The third term represents the shrinkage strain at time t , and the fourth term represents the strain caused by temperature gradients.

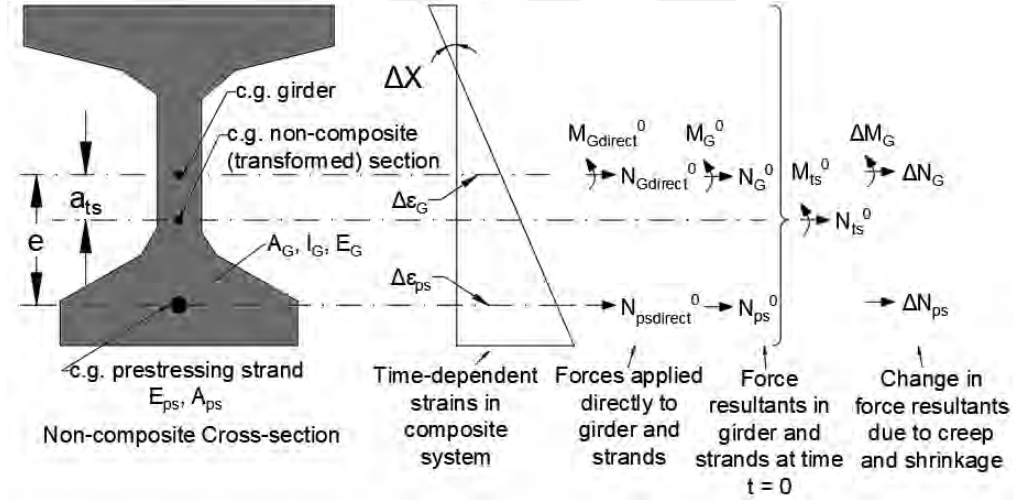


Figure 3.5. Distribution of forces in a prestressed concrete girder

Eqn. 3-20 uses an integral type creep law and uses the principle of superposition of stepwise prescribed stress histories. The principle of superposition to estimate the time-dependent deformation by time-varying stress history was first introduced by McHenry in 1943. He stated that the strain caused by stress increment at any time τ_i is independent from any stress applied either earlier or later. (Gilbert and Ranzi 2010). However, the stress increment at time $\tau_1 > \tau_0$ caused by time-dependent effects produces creep of smaller magnitude compared to a stress applied at time τ_0 , as illustrated in Figure 3.6. Therefore, Eqn. 3-20 implies that for each stress

increment between time intervals, creep coefficient should be recalculated, and using creep superposition law, the time-dependent strain should be determined.

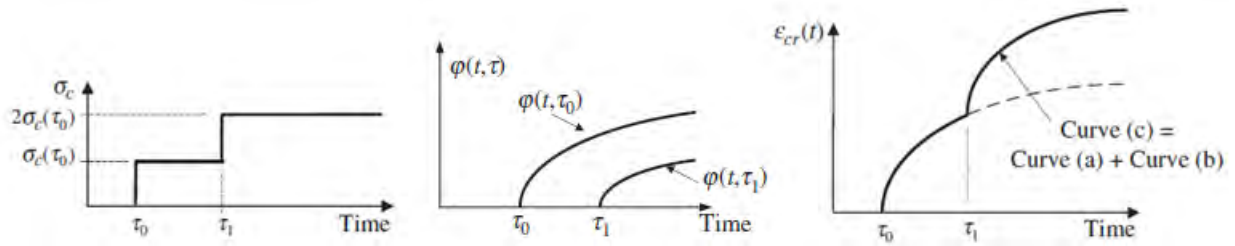


Figure 3.6. Principle of superposition for increasing stress increments (Gilbert and Ranzi 2010)

Gilbert and Ranzi (2010) also states that even though principle of superposition works well for increasing stress histories, as shown in Figure 3.6, for decreasing stress histories, it overestimates the recovery, as shown in Figure 3.7. However, this does not bring significant error in calculations if the decrease in the stress is not high when compared to initial stress, and for most practical purposes, and Eqn. 3-20 provides a reasonably good approximation of time-dependent strains for both increasing and decreasing stress history.

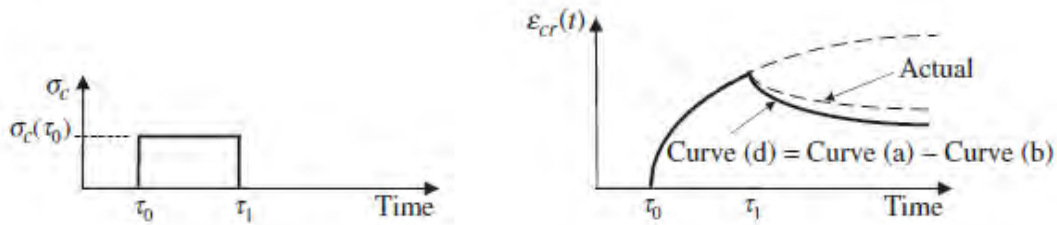
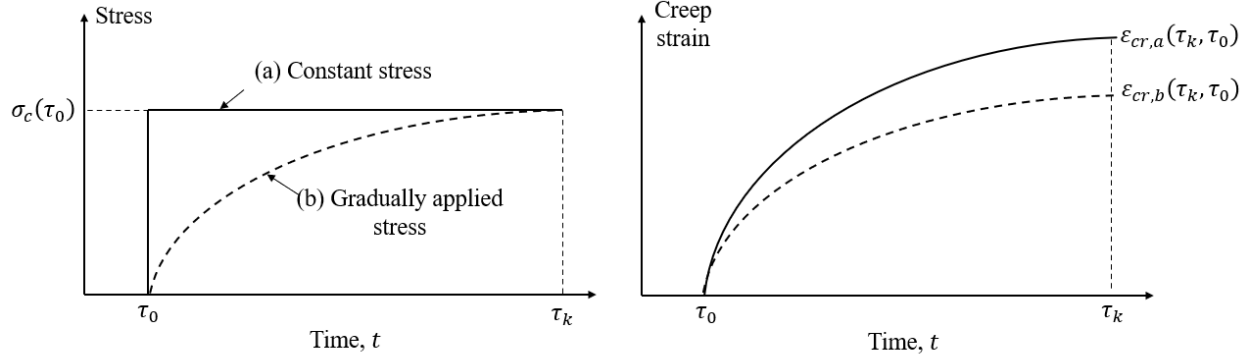


Figure 3.7. Principle of superposition for decreasing stress increments (Gilbert and Ranzi 2010)

Moreover, it is a fact that the creep strain also depends on how the load is applied. For instance, consider the two concrete stress histories in Figure 3.8. In stress history (a), a stress is applied at time τ_0 , and sustained until time τ_k . On the other hand, for stress history (b), the stress is gradually applied starting from time τ_0 , and reaching the exact same value at time τ_k . The creep strain caused by the stress history (a) is more than that resulting from the stress history (b). This is the phenomenon named as “ageing”.



$$\varepsilon_{cr,a}(\tau_k, \tau_0) = \frac{\sigma_c(\tau_0)}{E_c(\tau_0)} \varphi(\tau_k, \tau_0); \quad \varepsilon_{cr,b}(\tau_k, \tau_0) = \frac{\sigma_0(\tau_0)}{E_c(\tau_0)} \mu(\tau_k, \tau_0) \varphi(\tau_k, \tau_0);$$

Figure 3.8. Effect of loading type on creep strain (Gilbert and Ranzi 2010)

In a typical prestressed concrete beam, after strands are flame cut and prestressing force is transferred to the concrete, stresses in the concrete tend to decrease as a result of time-dependent effects such as creep and relaxation of strands, as illustrated in Figure 3.9. Besides, such decrease in stress in the concrete is gradual and small when compared to initially applied prestressing load. Hence, the stress increment $\Delta\sigma_c(t)$ can be treated as if it were introduced with its full magnitude at time t_0 and sustained to time t but the creep coefficient $\varphi(t, t_0)$ is replaced by a reduced value which equals $\mu\varphi(t, t_0)$, where μ is the ageing coefficient smaller than 1. (Granatta et al. 2013, Partov and Kantchev 2009, Ghali et al. 2002). With such simplification, Eqn. 3-20 can be rewritten as Eqn. 3-21.

$$\varepsilon_c(t_n) = \sigma_c(t_0) \frac{1 + \varphi(t_n, t_0)}{E_c(t_0)} + \Delta\sigma_c(t_n, t_0) \frac{1 + \mu(t_n, t_0)\varphi(t_n, t_0)}{E_c(t_0)} + \varepsilon_{sh}(t_n, t_0) + \varepsilon_{TG} \quad 3-21$$

Where:

$\Delta\sigma_c(t_n, t_0)$ is stress increment in concrete, gradually applied during the period t_0 to t_n ,

$\mu(t_n, t_0)$ is the ageing coefficient between t_0 and t_n .

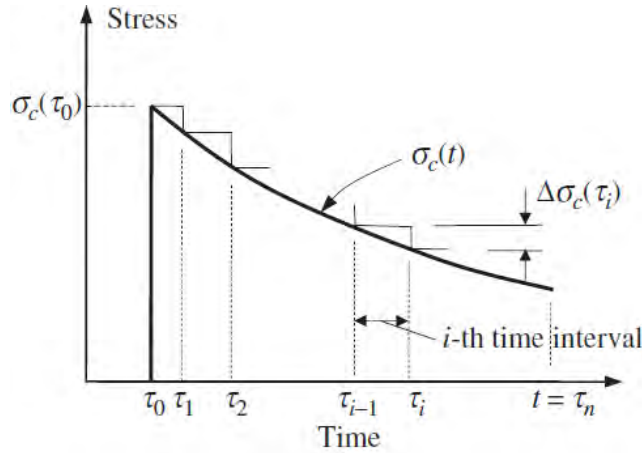


Figure 3.9. Representation of stress decrement with time (Gilbert and Ranzi 2010)

By introducing the parameter “ageing coefficient” into Eqn. 3-20, the analysis is greatly simplified such that the strain caused by stress increment can be directly determined without additional creep coefficient calculation as shown in Eqn. 3-21. Moreover, introducing the ageing coefficient into Eqn. 3-21 enables creep effects to be estimated using the age-adjusted effective modulus (AAEM) approach developed by Trost (1967), Dilger and Neville (1971), and Bazant (1972). The AAEM approach is used in cases when only a limited number of time steps are considered and an adjustment for the concrete age is made to account for: 1) the stiffening of the material with time, and 2) the change in concrete’s viscoelastic behavior as a function of various ages of loading.

In the AEMM approach, creep effects are modeled by varying the modulus of elasticity with time as a function of the creep coefficient and an ageing coefficient. For example, consider a concrete cylinder subject to a sustained load as in a typical creep test (Figure 3.10). Strain will initially increase proportionally with load until the load is kept constant at a certain magnitude (typically $0.4f'_c$). After the load is held, the cylinder will continue to deform (strain) due to creep. This increase in strain at constant stress can be captured by using a reduced modulus, which will vary with time. This time-varying modulus can be conveniently used to capture the effects of creep.

However, as the cylinder is loaded over time, creep effects are slightly counterbalanced by an increase in modulus due to the maturing concrete. As a result, to properly capture creep effects, the initial modulus is reduced with a creep coefficient, ϕ , but modified with an ageing coefficient, μ , representing the age-adjusted effective modulus, AEMM, approach.

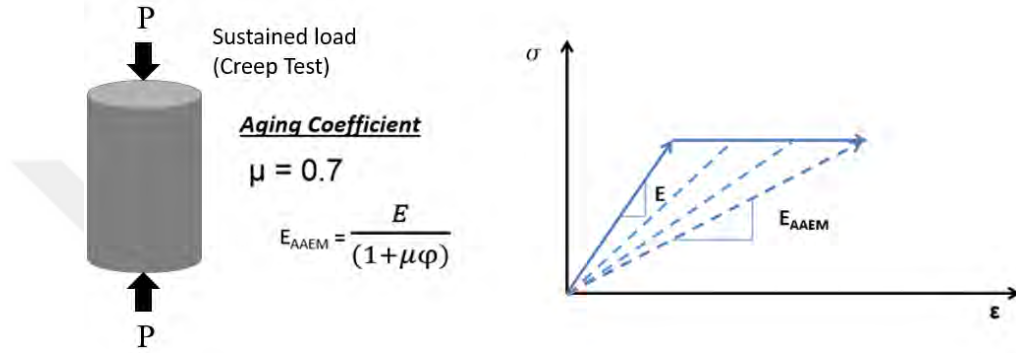


Figure 3.10. Illustration of age-adjusted effective modulus (AEMM) concept to capture creep effects

As a result, Eqn. 3-21 can be re-written as shown in Eqn. 3-22 using the effective and age adjusted effective modulus (EM, AAEM), respectively, which are defined in Eqns. 3-24 and 3-25. The assumed stress history shown in Figure 3.11a is idealized such that the initial stress and the change in stress are applied simultaneously over the considered time step (Figure 3.11b). The initial and creep induced strains due to initial stresses and changes in stress are illustrated in Figure 3.11c. The creep coefficient, ϕ , and the shrinkage strain, ϵ_{sh} , can be based on various existing models as mentioned in APPENDIX C.

$$\epsilon_c(t_n, t_0) = \frac{\sigma_c(t_0)}{E_{EM}(t_n, t_0)} + \frac{\Delta\sigma_c(t_n, t_0)}{E_{AAEM}(t_n, t_0)} + \epsilon_{sh}(t_n, t_0) + \epsilon_{TG} \quad 3-22$$

The first two terms in the Eqn. 3-22 can also be expressed as:

$$\epsilon_c(t_n) = [\epsilon_o + \epsilon_o\phi_o(t, t_o)] + [\Delta\epsilon_o + \Delta\epsilon_o\mu(t, t_o)\phi_o(t, t_o)] \quad 3-23$$

$$E_{EM}(t_n, t_0) = \frac{E_c(t_0)}{1 + \varphi(t_n, t_0)} \quad 3-24$$

$$E_{AAEM}(t_n, t_0) = \frac{E_c(t_0)}{1 + \mu(t_n, t_0)\varphi(t_n, t_0)} \quad 3-25$$

Where:

$E_{EM}(t, t_0)$ is the effective modulus,

$E_{AAEM}(t, t_0)$ is the age-adjusted effective modulus.

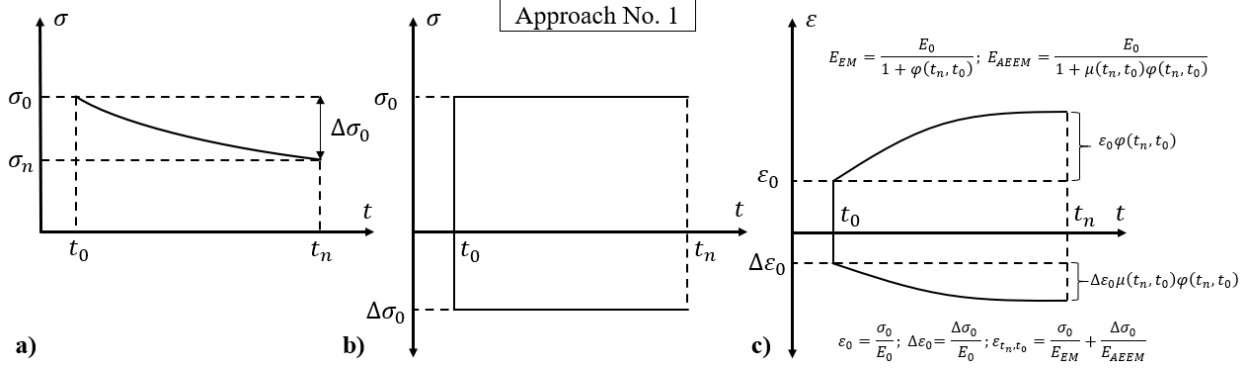


Figure 3.11. Illustration of Approach No. 1 for predicting time-dependent flexural deformations; a) assumed stress history, b) discretized stress history, and c) elastic and creep induced strains using Effective Modulus (EM) and Age-adjusted Effective Modulus (AAEM) approaches and based on discretized stress history

Typically, ageing coefficient depends on when the load is applied, that is, it is a time-dependent parameter (Ghali et al. 2002). In many handbooks, the derivation of the ageing coefficient is explained in detail, however, it is generally not of the prime concern in most practical design. Mostly, in practical computations, the ageing coefficient is either obtained from a chart or simply assumed a value between 0.6 and 0.9. In this approach, only one step is considered between times at prestress release and pre-erection. The ageing coefficient for girder is taken as 0.7 and initial modulus of elasticity, E_{ci} , is used in the calculations.

Moreover, the ageing coefficient can be eliminated by using effective modulus approach, EM. If the EM approach is used, a sufficient number of time steps should be considered together with a time-dependent function for the modulus to partially account for the effect of ageing. This is the approach used in Approaches No. 3, 4, and 5 which ignore the effect of loading age on concrete viscoelastic behavior, as well as in Approach No. 6, which takes into account the loading age by invoking separate creep curves in every time step.

As defined earlier, $\sigma(t_0)$ in Eqn. 3-22 represents initial stresses in the member. These stresses can be created by forces and moments that are initially directly applied to the precast girder or by forces and moments that are initially applied to the heterogeneous system (girder concrete and strands). It is assumed that the girder self-weight and prestressing force are applied to the heterogeneous system. To utilize the free shrinkage and creep properties of the precast beam, it is useful to decouple the internal forces acting on the heterogeneous section, into forces acting separately on the beam and strands.

The term $\Delta\sigma$ in Eqn. 3-22 represents the change in stress due to changes in axial forces and bending moments in the girder or prestressing strands. The changes in strains and stresses due to time-dependent effects can be calculated by using equations of material constitutive relationships, equilibrium and compatibility (Eqns. 3-26 - 3-31). For example, the change in axial strain at the centroid of the girder can be expressed by summing the creep strain due to initial axial forces, the elastic and creep strain due to the change in axial force over time and the free shrinkage strain (Eqn. 3-26). The change in curvature in the girder can be expressed by summing the creep curvature due to initial moments and the elastic and creep curvatures due to changes in these moments over time (Eqn. 3-27). The change in prestressing strand strain over time can be calculated by dividing the change in stress in the strand by the modulus of elasticity of the strand

(Eqn. 3-28). The change in stress in the strand over time represents either a prestress loss or prestress gain due to differential shrinkage and creep. Prestresses losses due to relaxation can be added to losses caused by creep and shrinkage to calculate the magnitude of the prestressing force at the end of the considered time step and at the beginning of the next time step.

In a statically determinate system, because there are no externally applied axial forces over time, the sum of changes in the axial forces in each component must equal zero (Eqn. 3-29). In addition, in such a determinate system because there are no externally applied bending moments over time, the sum of moment about the centroid of the girder must equal zero (Eqn. 3-30). Finally, assuming perfect bond between girder concrete and prestressing strands the axial strains at the centroid of each of these components can be inter-related by using the change in curvature and the distances between the centroids (Eqn. 3-31).

Equations 3-26 to 3-31 form a set of linear equations, which can be solved simultaneously to determine the 6 unknowns caused by the time-dependent effects $(\Delta\varepsilon_G, \Delta\varepsilon_{ps}, \Delta X, \Delta N_G, \Delta N_{ps}, \Delta M_G)$.

Material Constitutive Relationships

$$\varepsilon_{G(t_n, t_o)} = \frac{(N_{Gdirect}^0 + N_G^0)[\varphi_{G_o}(t_n, t_o)]}{E_{G_o} A_G} + \frac{\Delta N_G [1 + \mu_G(t_n, t_o)\varphi_{G_o}(t_n, t_o)]}{E_{G_o} A_G} + \varepsilon_{ShG}(t_n, t_o) \quad 3-26$$

$$\text{Where: } N_{Gdirect}^0 = 0, N_G^0 = \frac{A_G N_{ts}^0}{A_{ts_o}} - \frac{M_{ts}^0 a_{ts_o} A_G}{I_{ts_o}}; N_{ts}^0 = -P_e \text{ and } M_{ts}^0 = M_{Gsw} - P_e e_{tr}$$

$$\Delta X_{(t_n, t_o)} = \frac{(M_{Gdirect}^0 + M_G^0)[1 + \varphi_{G_o}(t_n, t_o)]}{E_{G_o} I_G} + \frac{\Delta M_G [1 + \mu_G(t_n, t_o)\varphi_{G_o}(t_n, t_o)]}{E_{G_o} I_G} \quad 3-27$$

$$\text{Where: } M_{Gdirect}^0 = 0, M_G^0 = \frac{M_{ts}^0 I_G}{I_{ts_o}}$$

$$\Delta \varepsilon_{ps(t_n, t_o)} = \frac{\Delta N_{ps}}{E_{ps} A_{ps}} \quad 3-28$$

Equilibrium

$$\Delta N_G + \Delta N_{ps} = 0 \quad 3-29$$

$$\Delta M_G + \Delta N_{ps} e = 0 \quad 3-30$$

Compatibility

$$\Delta \varepsilon_{ps(t_n, t_o)} = \Delta \varepsilon_{G(t_n, t_o)} + \Delta X_{(t_n, t_o)} e \quad 3-31$$

3.5.2. Approach No. 2

From this approach, the basic level of analysis was removed and only the advanced analysis was further developed since all calculations were conducted in Mathcad, and Matlab for Approaches No. 5 and 6, and there was no convenience to implement simplifying assumptions. Additionally, the approach used in the advanced analysis in Approach No. 1 was extended to predict camber and displacements in a composite bridge superstructure featuring prestressed concrete or steel beams. Only one time-step was considered between placement of deck and service life of the bridge. In addition, superimposed dead loads, such as barrier and overlay were applied shortly after the deck is placed. Therefore, no time-dependent deflection occurs between time at deck placement and time at the application of superimposed dead loads is considered. This assumption will not be valid after Approach No. 4 such that another time-dependent calculation will be performed between these two events. Changes in beam camber and displacements due to deck replacement were not considered and were within the scope of Approach No. 4 and further. In this approach, the effects of prestress, self-weight, beam shrinkage, prestress induced creep, self-weight induced creep, and differential shrinkage are considered as separate phenomena to

utilize the curvature diagram appropriate for each of these phenomena as presented in Figure 3.14. Using principles of second moment-area theorem with obtained change in curvature values for each phenomenon, the time-dependent changes in deflections for each factor were calculated, and they were simply summed up to obtain net change in deflection. This creates the inconvenience of conducting several strain compatibility based cross-sectional analyses at the ends and midspan for each phenomena to obtain the corresponding curvatures. This inconvenience is addressed in Approaches No. 5 and No. 6 when the effects of these phenomena are treated simultaneously, and numerical integration is used to compute camber and displacements. An algorithm written in Mathcad 15 was developed in which the user provides certain inputs, and the program calculates camber and displacements at the desired time. Two Mathcad files were prepared: The first was used to calculate camber and displacements in a prestressed concrete beam or a bridge that features such beams, and the second was used to predict service camber and displacements in a bridge that features steel girders. The algorithm considers the following five key time steps to compute time-dependent beam camber and displacements:

- 1) detensioning of the strands,
- 2) time from detensioning of the strands to deck placement,
- 3) deck placement,
- 4) application of superimposed dead loads (i.e. overlay and barriers) and opening of the bridge to traffic, and
- 5) time from opening of the bridge to traffic to the end of service life.

The approach is similar to that presented in Approach No. 1 except that there is a new component (i.e. the deck). Figure 3.12 shows the distribution of forces in a composite system consisting of a precast prestressed concrete girder and a cast-in-place deck or a steel girder and a

cast-in-place concrete deck. Since the analysis consists of a total of five time steps, three of which are discrete (strand detensioning, deck placement, and application of superimposed loads) while the other two vary in length, the ageing coefficient was taken equal to 0.7 for the time step between strand detensioning and deck placement, and 1.0 for the time step from deck placement to end of service life. This is done due to the fact that most of the maturing of concrete will take place during the time step from strand detensioning to deck placement. This of course does not take into account the effect of loading age on concrete creep behavior. When considering creep effects in the deck, the only time step that applies is the one from application of superimposed loads to end of bridge service life. Therefore, the ageing coefficient for the deck concrete was taken equal to 0.7 in this time step. The modulus of elasticity of girder concrete used in the time step from strand detensioning to deck placement is the initial modulus (i.e. the modulus when the strands are detensioned), whereas the one used from that time step is the 28 day modulus. Similarly, the modulus of elasticity for the deck concrete used between deck placement to final time is the initial modulus. Furthermore, after one time step is conducted, calculations for the next time step are performed by considering the updated forces and remaining creep coefficient and shrinkage strain.

Table 3.2. Values used in the analysis for ageing coefficient and modulus of elasticity

Time Interval	Ageing coefficient, μ		Modulus of elasticity, E	
	Girder	Deck	Girder	Deck
Strand detensioning - deck placement, $(t_o - t_d)$	0.7	-	E_{ci}	-
Deck placement - final time $(t_d - t_n)$	1.0	0.7	E_{c28}	E_{cdi}
E_{ci} = Modulus of elasticity for girder at release; E_{c28} = Modulus of elasticity of girder at 28 days; E_{cdi} = initial modulus of elasticity of the deck (calculated by assuming initial concrete strength of the deck is 80% of its 28 days strength)				

In this approach, the actual stress history shown in Figure 3.13a is discretized using two time steps (unlike in Approach No. 1, in which one-time step was used). The incurred changes in stress are separated between changes that occur during step 1 and those that occur during step 2.

The discretized stress history is shown in Figure 3.13b. The initial and creep induced strains due to initial stresses and changes in stress is illustrated in Figure 3.13c.

As defined earlier, $\sigma(t_0)$ in Eqn. 3-22 represent initial stresses in the member. These stresses can be created by forces and moments that are initially directly applied to the precast girder or the cast-in-place deck $(M_{Ddirect}^0, N_{Ddirect}^0, M_{Gdirect}^0, N_{Gdirect}^0, N_{psdirect}^0)$ or by forces and moments that are initially applied to the transformed or composite system $(N_{ts}^0, M_{ts}^0, N_c^0, M_c^0)$. For example, an eccentric prestressing force in a pre-tensioned girder creates axial forces and bending moments that are applied directly to the transformed section (girder and strands) in addition to the axial force applied directly to the prestressing strand. Similarly, the overlay loads such as barrier and overlay create axial forces and bending moments that are applied directly to the composite section (girder, deck and strands). To utilize the free shrinkage and creep properties of the precast concrete beam and cast-in-place concrete deck, it is useful to decompose the internal forces and moments acting on the transformed and/or composite section, into forces and moments acting separately on the girder and deck $(N_D^0, M_D^0, N_G^0, M_G^0, M_{ps}^0)$.

The term $\Delta\sigma$ in Eqn. 3-22 represents the change in stress due to changes in axial forces and bending moments in the deck, girder or prestressing strands. These are denoted as $\Delta M_D, \Delta N_D, \Delta M_G, \Delta N_G$ and ΔN_{ps} in Figure 3.12. The changes in strains and stresses due to time-dependent effects can be calculated by using equations of material constitutive relationships, equilibrium and compatibility (Eqns. 3-33 - 3-41 for composite prestressed concrete girders and Eqns. 3-42 - 3-48 for composite steel girders). For example, for the prestressed concrete girders, the change in axial strain at the centroid of the deck and girder can be expressed by summing the creep strain due to initial axial forces, the elastic and creep strain due to the change in axial force over time and the free shrinkage strain (Eqns. 3-33 & 3-34). For steel girders, the change in axial

strain at the centroid of the girder can be expressed by directly dividing the change in axial force in the girder by axial stiffness of the girder, EA (Eqn. 3-46). The change in curvature in the deck and prestressed girder can be expressed by summing the creep curvature due to initial moments and the elastic and creep curvatures due to changes in these moments over time (Eqns. 3-35 & 3-36). For the steel girders, the change in curvature in the girder can be expressed by directly dividing the change in moment in the girder by flexural stiffness of the girder, EI (Eqn. 3-47). For the prestressed concrete girder, the change in prestressing strand strain over time can be calculated by dividing the change in stress in the strand by the modulus of elasticity of the strand (Eqn. 3-37). The change in stress in the strand over time represents either a prestress loss or prestress gain due to differential shrinkage and creep. Prestresses losses due to relaxation can be added to losses caused by creep and shrinkage to calculate the magnitude of the prestressing force at the end of the considered time step and at the beginning of the next time step.

In a statically determinate system, because there are no externally applied axial forces over time, the sum of changes in the axial forces in each component must equal zero (Eqns. 3-38 and 3-44). In addition, in such a determinate system because there are no externally applied bending moments over time, the sum of moment about the centroid of the girder must equal zero (Eqns. 3-39 and 3-45). Finally, assuming perfect bond between deck concrete, girder concrete and prestressing strands (for prestressed concrete girders), the axial strains at the centroid of each of these components can be inter-related by using the change in curvature and the distances between the centroids (Eqns. 3-40 and 3-41, 3-48).

Equations 3-33 to 3-41 for composite prestressed concrete beams and Eqns. 3-42 to 3-48 for composite steel beams form sets of linear equations, which can be solved simultaneously to determine the 9 unknowns caused by the time-dependent effects in a composite prestressed

concrete beam system ($\Delta\epsilon_D, \Delta\epsilon_G, \Delta\epsilon_{ps}, \Delta X, \Delta N_D, \Delta N_G, \Delta N_{ps}, \Delta M_D, \Delta M_G$) and 7 unknowns in a composite steel girder system ($\Delta\epsilon_D, \Delta\epsilon_G, \Delta X, \Delta N_D, \Delta N_G, \Delta M_D, \Delta M_G$). For non-composite section analysis, Eqns. 3-26 to 3-31 can be used. Since they were already provided under Approach No. 1, they will not be repeated here.

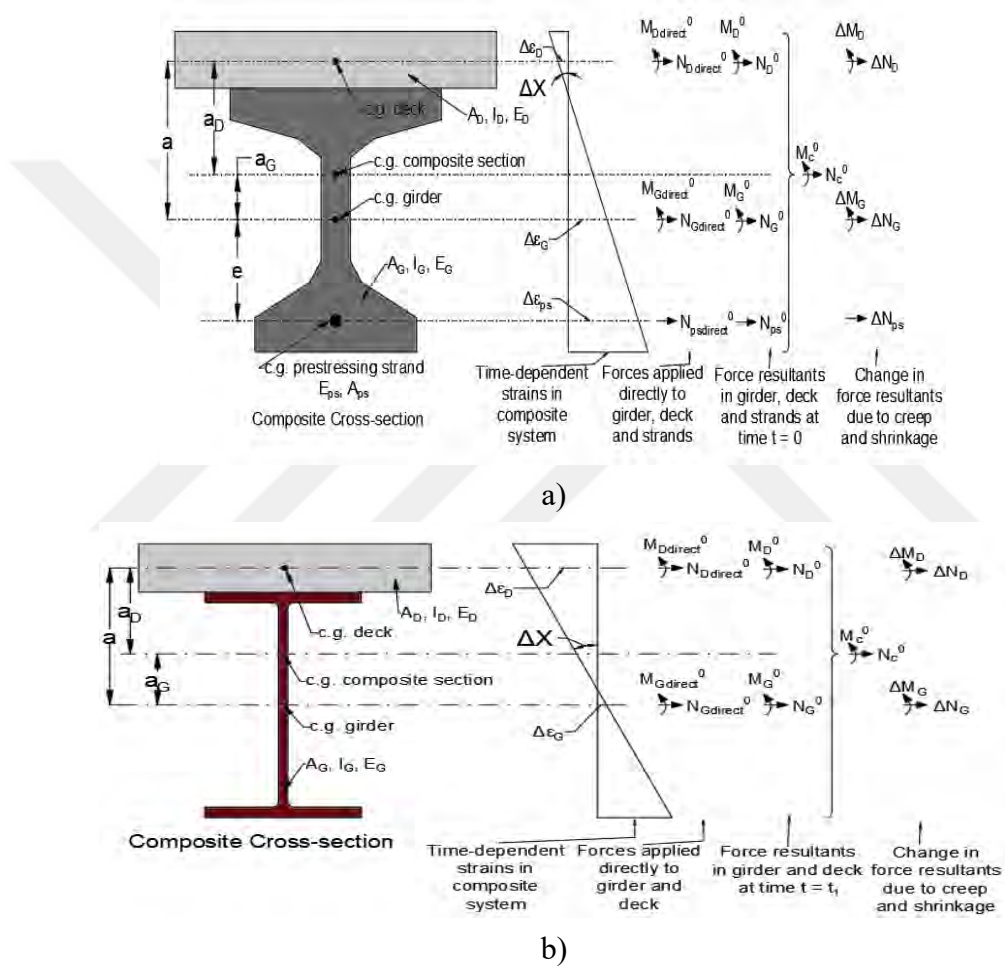


Figure 3.12. Distribution of forces in: a) composite prestressed concrete beam, and b) composite steel beam

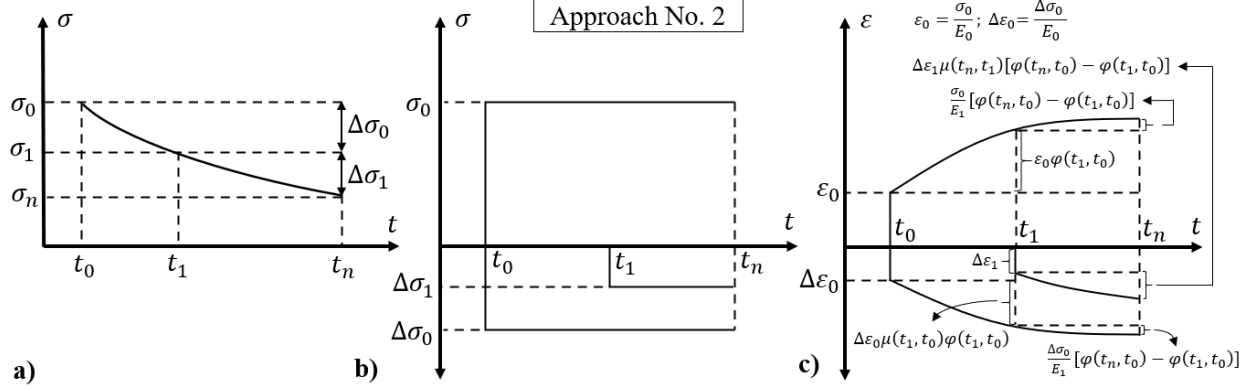


Figure 3.13. Illustration of Approach No. 2 for predicting time-dependent flexural deformations; a) assumed stress history, b) discretized stress history, and c) elastic and creep induced strains due to discretized stress history

Fundamental Equation

$$\begin{aligned} \varepsilon_{(t_n, t_o)} = & \left[\varepsilon_o + \varepsilon_o \varphi_o(t_1, t_o) + \frac{\sigma_o}{E_1} [\varphi_o(t_n, t_o) - \varphi_o(t_1, t_o)] \right] \\ & + \left[\Delta \varepsilon_o + \Delta \varepsilon_o \mu(t_1, t_o) \varphi_o(t_1, t_o) + \frac{\Delta \sigma_o}{E_1} [\varphi_o(t_n, t_o) - \varphi_o(t_1, t_o)] \right] \\ & + [\Delta \varepsilon_1 + \Delta \varepsilon_1 \mu(t_n, t_1) [\varphi_o(t_n, t_o) - \varphi_o(t_1, t_o)]] \end{aligned} \quad 3-32$$

Cross-sectional Analysis for Composite Prestressed Concrete Beams

$$\begin{aligned} \Delta \varepsilon_{G(t_n, t_d)} = & \frac{N_{G_d} [\varphi_{G_o}(t_n, t_o) - \varphi_{G_o}(t_d, t_o)]}{E_{G_d} A_G} \\ & + \frac{\Delta N_{G_d} [1 + \mu_G(t_n, t_d) [\varphi_{G_o}(t_n, t_o) - \varphi_{G_o}(t_d, t_o)]]}{E_{G_d} A_G} + [\varepsilon_{ShG}(t_n) - \varepsilon_{ShG}(t_d)] \end{aligned} \quad 3-33$$

Where: $N_{G_o} = N_{G_{direct}}^0 + N_G^0, N_{G_{direct}}^0 = 0, N_G^0 = \frac{A_G N_{ts}^0}{A_{ts_o}} - \frac{M_{ts}^0 a_{ts_o} A_G}{I_{ts_o}} + \frac{A_G N_c^0}{A_{c_o}} + \frac{M_c^0 a_{G_o} A_G}{I_{c_o}}$ and

$$N_{G_d} = N_{G_o} + \Delta N_{G_o}$$

$$\Delta \varepsilon_{D(t_n, t_d)} = \frac{(N_{D_{direct}}^0 + N_D^0) \varphi_{D_o}(t_n, t_d)}{E_{D_o} A_D} + \frac{\Delta N_D [1 + \mu_D(t_n, t_d) \varphi_{D_o}(t_n, t_d)]}{E_{D_o} A_D} + \varepsilon_{ShD}(t_n) \quad 3-34$$

$$\text{Where: } N_{Ddirect}^0 = 0, N_D^0 = \frac{n_o A_D N_c^0}{A_{c_o}} - \frac{M_c^0 a_{D_o} n_o A_D}{I_{c_o}}$$

$$\begin{aligned} \Delta X_{(t_n, t_d)} = & \frac{M_{G_d} [\varphi_{G_o}(t_n, t_o) - \varphi_{G_o}(t_d, t_o)]}{E_{G_d} I_G} \\ & + \frac{\Delta M_{G_d} [1 + \mu_G(t_n, t_d) [\varphi_{G_o}(t_n, t_o) - \varphi_{G_o}(t_d, t_o)]]}{E_{G_d} I_G} \end{aligned} \quad 3-35$$

$$\text{Where: } M_{G_o} = M_{Gdirect}^0 + M_G^0, M_{Gdirect}^0 = 0, M_G^0 = \frac{M_{ts}^0 I_G}{I_{ts_o}} + \frac{I_G M_c^0}{I_{c_o}} \text{ and } M_{G_d} = M_{G_o} + \Delta M_{G_o}$$

$$\Delta X_{(t_n, t_d)} = \frac{(M_{Ddirect}^0 + M_D^0) \varphi_{D_o}(t_n, t_d)}{E_{D_o} I_D} + \frac{\Delta M_D [1 + \mu_D(t_n, t_d) \varphi_{D_o}(t_n, t_d)]}{E_{D_o} I_D} \quad 3-36$$

$$\text{Where: } M_{Ddirect}^0 = 0, M_D^0 = \frac{M_c^0 n_o I_D}{I_{c_o}}$$

$$\text{Note: } N_{ts}^0 = -P_e, M_{ts}^0 = M_{Gsw} + M_{dsw} - P_e e_{tr}, N_c^0 = 0, M_c^0 = M_{bsw} + M_{osw}$$

$$\Delta \varepsilon_{ps(t_n, t_d)} = \frac{\Delta N_{ps}}{E_{ps} A_{ps}} \quad 3-37$$

$$\Delta N_{G_d} + \Delta N_D + \Delta N_{ps} = 0 \quad 3-38$$

$$\Delta M_{G_d} + \Delta M_D - \Delta N_D a + \Delta N_{ps} e = 0 \quad 3-39$$

$$\Delta \varepsilon_{D(t_n, t_d)} = \Delta \varepsilon_{G(t_n, t_d)} - \Delta X_{(t_n, t_d)} a \quad 3-40$$

$$\Delta \varepsilon_{ps(t_n, t_d)} = \Delta \varepsilon_{G(t_n, t_d)} + \Delta X_{(t_n, t_d)} e \quad 3-41$$

Cross-sectional Analysis for Composite Steel Beams

$$\Delta \varepsilon_{D(t_n, t_d)} = \frac{(N_{Ddirect}^0 + N_D^0) \varphi_{D_o}(t_n, t_d)}{E_{D_o} A_D} + \frac{\Delta N_D [1 + \mu_D(t_n, t_d) \varphi_{D_o}(t_n, t_d)]}{E_{D_o} A_D} + \varepsilon_{shD}(t_n) \quad 3-42$$

$$\text{Where: } N_{Ddirect}^0 = 0, N_D^0 = \frac{n_o A_D N_c^0}{A_{c_o}} - \frac{M_c^0 a_{D_o} n_o A_D}{I_{c_o}}$$

$$\Delta X_{(t_n, t_d)} = \frac{(M_{Ddirect}^0 + M_D^0) \varphi_{D_o}(t_n, t_d)}{E_{D_o} I_D} + \frac{\Delta M_D [1 + \mu_D(t_n, t_d) \varphi_{D_o}(t_n, t_d)]}{E_{D_o} I_D} \quad 3-43$$

$$\text{Where: } M_{Ddirect}^0 = 0, M_D^0 = \frac{M_c^0 n_o I_D}{I_{c_o}}$$

$$\text{Note: } N_c^0 = 0, M_c^0 = M_{bsw} + M_{osw}$$

$$\Delta N_D + \Delta N_{ps} = 0 \quad 3-44$$

$$\Delta M_G + \Delta M_D - \Delta N_D a = 0 \quad 3-45$$

$$\Delta \varepsilon_{G(t_n, t_d)} = \frac{\Delta N_G}{E_G A_G} \quad 3-46$$

$$\Delta X_{(t_n, t_d)} = \frac{\Delta M_G}{E_G I_G} \quad 3-47$$

$$\Delta \varepsilon_{D(t_n, t_d)} = \Delta \varepsilon_{G(t_n, t_d)} - \Delta X_{(t_n, t_d)} a \quad 3-48$$

It should be noted that for the prestressed concrete girders, prestresses losses due to relaxation of strands should be added to losses caused by creep and shrinkage to calculate the magnitude of the prestressing force at the end of first time step (i.e. at deck placement) and forces should be updated at the beginning of next time step.

The initial curvatures and the change in curvature obtained using Eqns. 3-35 and 3-43 can be used to obtain camber and displacements at the beginning of the time step as well as at the end of it using the moment area method and the applicable curvature diagrams, as presented in Figure 3.14. Unlike previous approach, this approach allows the use of various harping points. Curvatures are calculated based on the transformed section properties for both the non-composite and

composite section. Prestress and prestress creep create negative curvatures. Beam shrinkage in the non-composite section and deck shrinkage in the composite section create positive curvatures, whereas beam shrinkage in the composite section creates negative curvatures. The self-weight of beam and deck and the creep due to self-weight create positive curvatures. Although these curvature diagrams can be used to calculate camber and displacement at any point in time using the moment area theorems, only midspan camber was reported in this approach. The proposed method for prestressed concrete girders can be summarized as follows:

- Perform time-dependent analysis at the cross-sectional level for each time step assuming the beam is simply supported.
 - ❖ Calculate curvatures for each time step at midspan and ends of beam
 - ✓ Time Step 1: Calculate curvatures due to the effects of the prestressing force and self-weight using transformed section properties.
 - ✓ Time Step 2: Calculate time-dependent effects (such as changes in forces and curvatures) at the cross-sectional level using transformed section properties (beam and strand) due to forces computed in Time Step 1.
 - ✓ Time Step 3: Calculate instantaneous curvatures due to deck weight and superimposed dead loads.
 - ✓ Time Step 4: Calculate time-dependent effects at a sectional level using transformed section properties (beam and deck and strand) due to forces computed in the previous time steps.
 - ❖ Draw curvature diagram for the beam/superstructure at each time step

- ❖ Calculate camber and displacements using the second moment area theorem at each time step



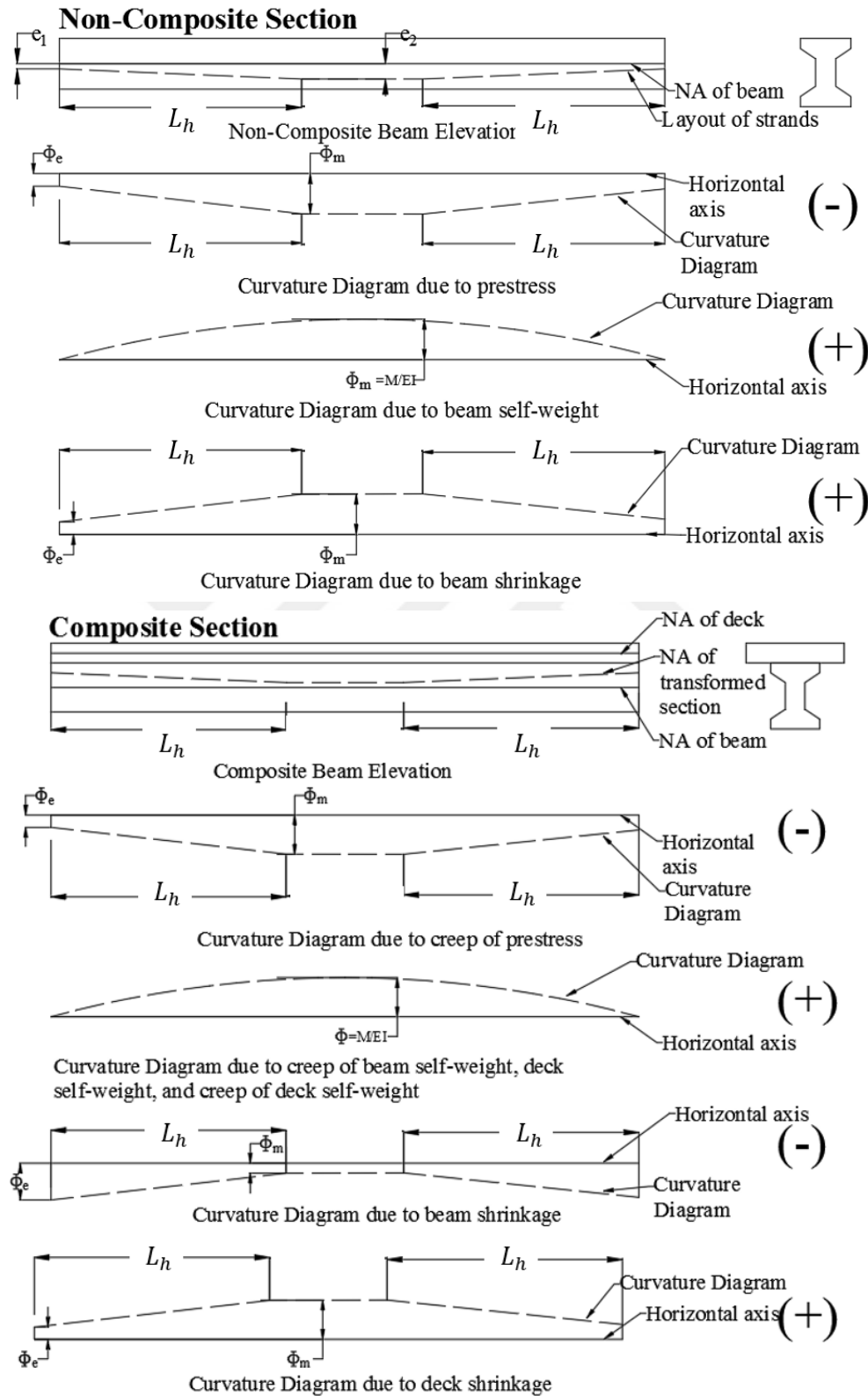


Figure 3.14. Curvature diagrams for composite and non-composite sections (Menkulasi 2018)

3.5.3. Approach No. 3

This approach uses the Effective Modulus (EM) method as opposed to the Age-adjusted Effective Modulus (AAEM) method used in the previous approaches. In the EM method, additional time substeps are considered between major construction activities, shown *above*, that constitute the major time steps used in the AAEM method. To quantify the time-dependent effects for each time substep, remaining creep coefficients and shrinkage strains are utilized. Additionally, a time-dependent function for the modulus is adopted so that the appropriate modulus can be used for each time step. This approach partially removes the necessity to use an ageing coefficient since the modulus of elasticity is updated continuously throughout the analysis using the adopted time-dependent function. The need for an ageing coefficient is not entirely removed since a single creep curve is used for the beam and another for the deck concrete. Therefore, the effect of loading age on creep behavior is ignored. Additionally, the utilization of additional time substeps theoretically improves camber and displacement predictions for the next time step since initial stress conditions are continuously updated. However, the impact of this improvement was not clear, and this approach provided an opportunity to quantify the difference between predictions using this approach and that used in Approach No. 2. The strain compatibility based cross-sectional analysis is conducted in an identical manner with that presented in the previous approach with the exception that the ageing coefficient is set equal to 1.0. Similar to previous approach, each factor such as girder self-weight induced creep effect and prestressing induced creep effect is considered separately. After calculating time-dependent changes in curvatures at midspan and end section for each factor, the net change in deflection can be calculated using moment-area theorem with given curvature diagrams shown in Figure 3.14. This approach is also implemented in Mathcad and the user is asked to provide input related to girder properties, deck properties, prestressing steel properties, and environmental conditions at the precast facility or bridge site. Then, the user is

asked to select the desired creep and shrinkage model to be used in the time-dependent analysis. Finally, the program also has the ability to calculate camber and displacements caused by temperature gradients for beams featuring certain geometries.

The strain at any given fiber in composite or non-composite cross-sections can be calculated using Eqn. 3-49. This is illustrated in Figure 3.15. In this approach, the actual stress history shown in Figure 3.15a is discretized using multiple timesteps (unlike in Approach No. 1 and Approach No. 2, in which one and two time steps were used, respectively). The incurred changes in stress are separated between changes that occur between a time step. The discretized stress history is shown in Figure 3.15b. The initial and creep induced strains due to initial stresses and changes in stress is illustrated in Figure 3.15c. The set of equations that can be used to conduct time-dependent cross-sectional analysis for non-composite cross-sections, composite cross-sections featuring a prestressed concrete beam, and composite cross-sections featuring a steel beam are provided in Eqns. 3-50 - 3-55, Eqns. 3-56 - 3-64, and Eqns. 3-65 - 3-71, respectively. These sets of equations apply also to Approach No. 4 and Approach No. 5, although there are differences in the way that they are implemented. These differences are explained in the subsequent sections.

Fundamental Equation

$$\begin{aligned} \varepsilon_{(t_n, t_o)} = \varepsilon_o + \sum_{j=0}^m \left[\frac{\sigma_o}{E_j} [\varphi_o(t_{j+1}, t_o) - \varphi_o(t_j, t_o)] \right. \\ \left. + \left[\Delta\varepsilon_j + \sum_{i=j}^m \frac{\Delta\sigma_j}{E_i} [\varphi_o(t_{i+1}, t_o) - \varphi_o(t_i, t_o)] \right] \right] \end{aligned} \quad 3-49$$

$$\varepsilon_i = \frac{\sigma_i}{E_i}; n = \text{number of time steps}; m = \text{number of loading events}; n = m + 1$$

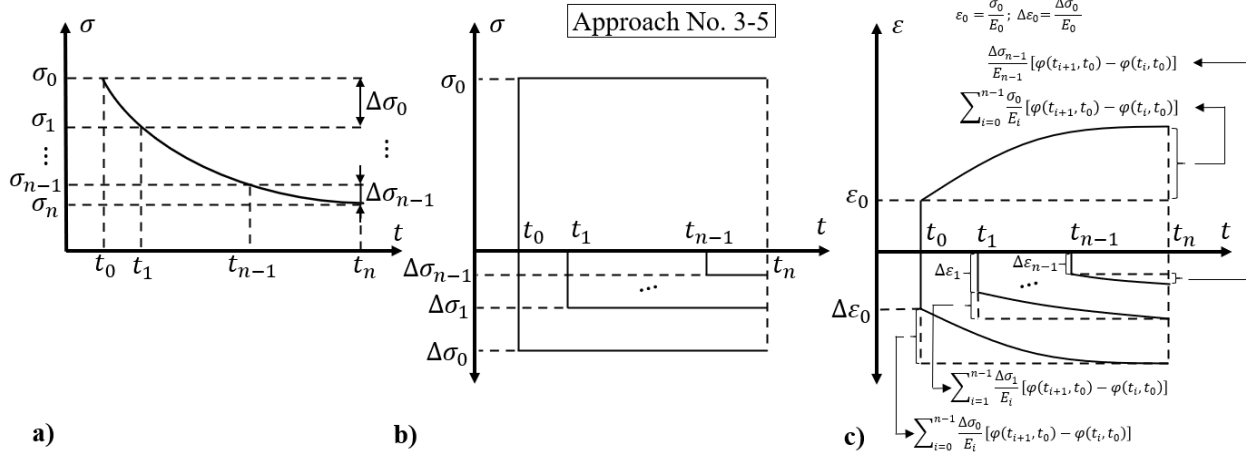


Figure 3.15. Illustration of Approach No. 3-5 for predicting time-dependent flexural deformations; a) assumed stress history, b) discretized stress history, and c) elastic and creep induced strains due to discretized stress history

Cross-sectional Analysis for Non-composite (i.e. Transformed) Prestressed Concrete Beams

$$\Delta \varepsilon_{G(t_n, t_0)} = \sum_{i=0}^m \left[\frac{N_{G_i} [\varphi_{G_0}(t_{i+1}, t_0) - \varphi_{G_0}(t_i, t_0)]}{E_{G_i} A_G} + \frac{\Delta N_{G_i} [1 + [\varphi_{G_0}(t_{i+1}, t_0) - \varphi_{G_0}(t_i, t_0)]]}{E_{G_i} A_G} + [\varepsilon_{ShG}(t_{i+1}) - \varepsilon_{ShG}(t_i)] \right] \quad 3-50$$

Where: $N_{G_0} = N_{G_{direct}}^0 + N_G^0$, $N_{G_{direct}}^0 = 0$, $N_G^0 = \frac{N_{ts}^0 A_G}{A_{ts_0}} - \frac{M_{ts}^0 a_{ts_0} A_G}{I_{ts_0}}$ and

$$N_{G_{i+1}} = N_{G_i} + \Delta N_{G_i}$$

$$\Delta X_{(t_n, t_0)} = \sum_{i=0}^m \left[\frac{M_{G_i} [\varphi_{G_0}(t_{i+1}, t_0) - \varphi_{G_0}(t_i, t_0)]}{E_{G_i} I_G} + \frac{\Delta M_{G_i} [1 + [\varphi_{G_0}(t_{i+1}, t_0) - \varphi_{G_0}(t_i, t_0)]]}{E_{G_i} I_G} \right] \quad 3-51$$

Where: $M_{G_0} = M_{G_{direct}}^0 + M_G^0$, $M_{G_{direct}}^0 = 0$, $M_G^0 = \frac{M_{ts}^0 I_G}{I_{ts_0}}$ and $M_{G_{i+1}} = M_{G_i} + \Delta M_{G_i}$

Note: $N_{ts}^0 = -P_e, M_{ts}^0 = M_{Gsw} - P_e e_{tr}$

$$\Delta \varepsilon_{ps(t_n, t_o)} = \frac{\sum_{i=0}^m \Delta N_{ps_i}}{E_{ps} A_{ps}} \quad 3-52$$

$$\Delta N_{G_i} + \Delta N_{ps_i} = 0 \quad 3-53$$

$$\Delta M_{G_i} + \Delta N_{ps_i} e = 0 \quad 3-54$$

$$\Delta \varepsilon_{ps(t_n, t_o)} = \Delta \varepsilon_{G(t_n, t_o)} + \Delta X_{(t_n, t_o)} e \quad 3-55$$

Cross-sectional Analysis for Composite Prestressed Concrete Beams

$$\Delta \varepsilon_{G(t_n, t_d)} = \sum_{i=r}^m \left[\frac{N_{G_i} [\varphi_{G_o}(t_{i+1}, t_o) - \varphi_{G_o}(t_i, t_o)]}{E_{G_i} A_G} + \frac{\Delta N_{G_i} [1 + [\varphi_{G_o}(t_{i+1}, t_o) - \varphi_{G_o}(t_i, t_o)]]}{E_{G_i} A_G} + [\varepsilon_{ShG}(t_{i+1}) - \varepsilon_{ShG}(t_i)] \right] \quad 3-56$$

Where: $N_{G_0} = N_{G_{direct}}^0 + N_G^0, N_{G_{direct}}^0 = 0, N_G^0 = \frac{N_{ts}^0 A_G}{A_{ts_0}} - \frac{M_{ts}^0 a_{ts_0} A_G}{I_{ts_0}},$

$$N_{G_r} = N_{G_0} + \frac{N_c^0 A_G}{A_{c_0}} + \frac{M_c^0 a_{G_0} A_G}{I_{c_0}} + \left[\sum_{i=0}^{r-1} \Delta N_{G_i} \right] \text{ and } N_{G_{i+1}} = N_{G_i} + \Delta N_{G_i}$$

$$\Delta \varepsilon_{D(t_n, t_d)} = \sum_{i=r}^m \left[\frac{N_{D_i} [\varphi_{D_o}(t_{i+1}, t_d) - \varphi_{D_o}(t_i, t_d)]}{E_{D_i} A_D} + \frac{\Delta N_{D_i} [1 + [\varphi_{D_o}(t_{i+1}, t_d) - \varphi_{D_o}(t_i, t_d)]]}{E_{D_i} A_G} + [\varepsilon_{ShD}(t_{i+1}) - \varepsilon_{ShD}(t_i)] \right] \quad 3-57$$

Where: $N_{D_0} = N_{D_{direct}}^0 + N_D^0, N_{D_{direct}}^0 = 0, N_D^0 = \frac{N_c^0 n_0 A_D}{A_{c_0}} - \frac{M_c^0 a_{D_0} n_0 A_D}{I_{c_0}},$

$$N_{D_r} = N_{D_0} \text{ and } N_{D_{i+1}} = N_{D_i} + \Delta N_{D_i}$$

$$\Delta X_{(t_n, t_d)} = \sum_{i=r}^m \left[\frac{M_{G_i} [\varphi_{G_o}(t_{i+1}, t_o) - \varphi_{G_o}(t_i, t_o)]}{E_{G_i} I_G} + \frac{\Delta M_{G_i} [1 + [\varphi_{G_o}(t_{i+1}, t_o) - \varphi_{G_o}(t_i, t_o)]]}{E_{G_i} I_G} \right] \quad 3-58$$

$$\text{Where: } M_{G_0} = M_{G_{direct}}^0 + M_G^0, M_{G_{direct}}^0 = 0, M_G^0 = \frac{M_{ts}^0 I_G}{I_{ts_0}},$$

$$M_{G_r} = M_{G_0} + \frac{M_G^0 I_G}{I_{c_0}} + \left[\sum_{i=0}^{r-1} \Delta M_{G_i} \right] \text{ and } M_{G_{i+1}} = M_{G_i} + \Delta M_{G_i}$$

$$\Delta X_{(t_n, t_d)} = \sum_{i=r}^m \left[\frac{M_{D_i} [\varphi_{D_o}(t_{i+1}, t_d) - \varphi_{D_o}(t_i, t_d)]}{E_{D_i} I_D} + \frac{\Delta M_{D_i} [1 + [\varphi_{D_o}(t_{i+1}, t_d) - \varphi_{D_o}(t_i, t_d)]]}{E_{D_i} I_D} \right] \quad 3-59$$

$$\text{Where: } M_{D_0} = M_{D_{direct}}^0 + M_D^0, M_{D_{direct}}^0 = 0, M_D^0 = \frac{M_c^0 n_0 I_D}{I_{c_0}},$$

$$M_{D_r} = M_{D_0} \text{ and } M_{D_{i+1}} = M_{D_i} + \Delta M_{D_i}$$

$$\text{Note: } N_{ts}^0 = -P_e, M_{ts}^0 = M_{G_{sw}} + M_{d_{sw}} - P_e e_{tr}, N_c^0 = 0, M_c^0 = M_{b_{sw}} + M_{osw}$$

$$\Delta \varepsilon_{ps(t_n, t_d)} = \frac{\sum_{i=r}^m \Delta N_{ps_i}}{E_{ps} A_{ps}} \quad 3-60$$

$$\Delta N_{D_i} + \Delta N_{G_i} + \Delta N_{ps_i} = 0 \quad 3-61$$

$$\Delta M_{G_i} + \Delta M_{D_i} - \Delta N_{D_i} a + \Delta N_{ps_i} e = 0 \quad 3-62$$

$$\Delta \varepsilon_{D(t_n, t_d)} = \Delta \varepsilon_{G(t_n, t_d)} - \Delta X_{(t_n, t_d)} a \quad 3-63$$

$$\Delta \varepsilon_{ps(t_n, t_d)} = \Delta \varepsilon_{G(t_n, t_d)} + \Delta X_{(t_n, t_d)} e \quad 3-64$$

Cross-sectional Analysis for Composite Steel Beams

$$\begin{aligned} \Delta \varepsilon_{D(t_n, t_d)} = \sum_{i=r}^m & \left[\frac{N_{D_i} [\varphi_{D_o}(t_{i+1}, t_d) - \varphi_{D_o}(t_i, t_d)]}{E_{D_i} A_D} \right. \\ & + \frac{\Delta N_{D_i} (1 + [\varphi_{D_o}(t_{i+1}, t_d) - \varphi_{D_o}(t_i, t_d)])}{E_{D_i} A_G} \\ & \left. + [\varepsilon_{ShD}(t_{i+1}) - \varepsilon_{ShD}(t_i)] \right] \end{aligned} \quad 3-65$$

Where: $N_{D_0} = N_{D_{direct}}^0 + N_D^0, N_{D_{direct}}^0 = 0, N_D^0 = \frac{N_c^0 n_0 A_D}{A_{c_0}} - \frac{M_c^0 a_{D_0} n_0 A_D}{I_{c_0}},$

$$N_{D_r} = N_{D_0} \text{ and } N_{D_{i+1}} = N_{D_i} + \Delta N_{D_i}$$

$$\begin{aligned} \Delta X_{(t_n, t_d)} = \sum_{i=r}^m & \left[\frac{M_{D_i} [\varphi_{D_o}(t_{i+1}, t_d) - \varphi_{D_o}(t_i, t_d)]}{E_{D_i} I_D} \right. \\ & \left. + \frac{\Delta M_{D_i} [1 + [\varphi_{D_o}(t_{i+1}, t_d) - \varphi_{D_o}(t_i, t_d)]]}{E_{D_i} I_D} \right] \end{aligned} \quad 3-66$$

Where: $M_{D_0} = M_{D_{direct}}^0 + M_D^0, M_{D_{direct}}^0 = 0, M_D^0 = \frac{M_c^0 n_0 I_D}{I_{c_0}},$

$$M_{D_r} = M_{D_0} \text{ and } M_{D_{i+1}} = M_{D_i} + \Delta M_{D_i}$$

Note: $N_c^0 = 0, M_c^0 = M_{bsw} + M_{osw}$

$$\Delta X_{(t_n, t_d)} = \frac{\Delta M_G}{E_G I_G} \quad 3-67$$

$$\Delta \varepsilon_{G(t_n, t_d)} = \frac{\Delta N_G}{E_G A_G} \quad 3-68$$

$$\Delta N_{D_i} + \Delta N_{G_i} = 0 \quad 3-69$$

$$\Delta M_{G_i} + \Delta M_{D_i} - \Delta N_{D_i} a = 0 \quad 3-70$$

$$\Delta \varepsilon_{D(t_n, t_d)} = \Delta \varepsilon_{G(t_n, t_d)} - \Delta X_{(t_n, t_d)} a \quad 3-71$$

It should be noted that for the prestressed concrete girders, prestresses losses due to relaxation should be added to losses caused by creep and shrinkage to calculate the magnitude of the prestressing force at the end of the considered time step and the forces should be updated at the beginning of the next time step.

3.5.4. Approach No. 4

In this approach, the EM method used in Approach No. 3 was extended to capture the effects of deck replacement on beam camber and displacement. In addition to this, unlike previous approaches, Approach No. 2 and Approach No. 3, it is assumed that time-dependent change in camber and displacements occurs between the time at deck placement and at application of superimposed dead loads. This is more realistic because in most construction activities, decks are cured for some days and after curing period ends, overlay and barrier loads are applied to the composite structure. Between these two events, it is highly possible to have some time-dependent change in camber. This is illustrated in section 8.6. Apart from these two, the methodology for estimating time-dependent deflections and procedure is same with Approach No. 3.

To model the effect of deck removal and the corresponding beam rebound, the loading history was modified to include unloading steps for the self-weight of existing barrier, existing overlay, existing deck, and locked in forces in the deck, as well as new loading steps for the weight

of the new deck, weight of new overlay, and weight of new barrier. The removal of the self-weight of the deck and the removal of internal forces present in the deck were treated as new forces applied at the time of deck removal to the non-composite beam section. The removal of barrier and overlay were considered as new forces applied to the composite system. The calculation of the immediate rebound involves calculating the upward displacement (camber) caused by the removal of existing barrier, overlay, and deck weight, as well as the removal of internal axial forces and moments in the existing deck. The calculation of longer-term rebound requires a time-dependent analysis. Time-dependent effects due to these newly applied forces were calculated using the remaining creep coefficient for the beam concrete based on a creep curve with a loading time of 1 day (i.e. the application of prestressing force when the beams are fabricated). This approach, similar to the approaches presented so far, relies on the use of a single creep curve for beam and deck concrete. In this approach, each phenomenon that causes time-dependent camber and displacements is treated separately and the strain compatibility based cross-sectional analysis is based on two sections: beam end and midspan. For each phenomenon, the corresponding curvature diagrams are used to predict camber and displacements at midspan.

A total of twelve key time steps were considered for prestressed concrete girders to compute time-dependent camber and displacements. These time steps include:

- 1) detensioning of the strands,
- 2) time from detensioning of the strands to deck placement,
- 3) time at which the deck is placed,
- 4) time from deck placement to placement of barriers, overlay (assumed to be 7 days in the analyses),
- 5) time at which the barriers and overlay are placed and the opening of bridge to traffic,

- 6) time from the opening of the bridge to traffic to deck removal for projects that features a deck replacement (assumed to be 50 years in the analyses),
- 7) time at which the old deck is removed,
- 8) time from deck removal to new deck placement (assumed to be 10 days in the analyses),
- 9) time at which the new deck is placed,
- 10) time from new deck placement to placement of new barrier and overlay (assumed to be 7 days in the analyses),
- 11) time at which the barriers and overlay are placed and the reopening of bridge to traffic,
- 12) time from reopening of bridge to traffic to the end of service life, which is considered to be 75 years.

In this approach, the user has the ability to enter time in days at discrete time steps such that camber and displacements are calculated at the desired times. Additionally, because camber and displacements at a given time step depend on the conditions in the previous time step, such as the predicted camber and forces in the previous time step, the user has the ability to enter the camber in the previous time step if such information is available. In this case, the algorithm overrides the predicted camber in the previous time step with that specified by the user. When camber and displacements in a steel bridge are desired, fabricated camber must be provided as an input. Then the effect of differential shrinkage between the deck and the steel girder is considered to calculate service camber and displacements.

3.5.5. Approach No. 5

In this approach, the computer algorithm prepared in Mathcad was translated into Matlab and was improved to develop a graphical user interface, and was further enhanced to improve the accuracy of camber and displacement predictions. Unlike the previous approaches, in this

approach the strain compatibility based cross-sectional analysis is conducted for several sections along the length the beam rather than only at midspan and beam ends. The calculated cross-sectional curvatures are then used to construct a curvature diagram, and numerical integration together with the second moment area theorem is used to compute camber and displacements at any location along the span, as explained in *above*. By this way, the deformed shape of the beam at any time can be graphically described. In this approach, the effects of creep and shrinkage (element based as well as differential), as well as steel relaxation are considered simultaneously during the strain compatibility based cross-sectional analysis. This is different from the approach used in the previous approaches in which the effects of these phenomena were considered separately to utilize the curvature diagrams appropriate for each phenomenon. The utilization of numerical integration removes the need to rely on curvature diagrams for which existing deflection formulations can be employed to predict camber and displacements. The flowchart used to create the code for this approach is provided in Figure 3.16.

The algorithm starts with the creation of arrays for number of time steps and number of sections. Then time-dependent material properties such as the modulus of elasticity, creep, and shrinkage are calculated. Similarly, time-dependent section properties are calculated since the calculation of composite section properties depends on the modular ratio (E_{ps}/E_c or E_{deck}/E_c). Then, initial curvatures, axial forces, bending moments, and deflections caused by self-weight and the prestressing force are calculated. The algorithm operates based on two loops. The first represents the number of time steps at which camber and displacements are desired to be calculated. The second, which runs within the first loop, represents the number of sections along the beam. For a given time step a strain compatibility based cross-sectional analysis is conducted for multiple sections along the length of the beam with the purpose of constructing a curvature

diagram and computing deflections by using numerical integration and second moment area theorem. The change in axial force, bending moment, and strand strain is recorded, and this information is used to update initial conditions for next time step. This procedure is repeated until all desired time steps are considered. The computed camber and displacement can then be plotted as a function of time.

Another improvement offered by Approach No. 5, is the ability to calculate the deflected shape of the beam as a function of time. The graphical user interface offers the user the ability to plot camber/displacements as a function of time and location along the beam span from beam end to midspan. In this approach, just like in the previous approaches, axial forces and moments are summed at the end of each time step so that time-dependent effects can be calculated for the next time step based on a single creep curve for beam and deck concrete with loading times that correspond with prestress release and deck placement, respectively. The set of equations that can be used to conduct time-dependent cross-sectional analysis for non-composite cross-sections, composite cross-sections featuring a prestressed concrete beam, and composite cross-sections featuring a steel beam are provided in Eqns. 3-50 - 3-55, Eqns. 3-56 - 3-64, and Eqns. 3-65 - 3-71, respectively.

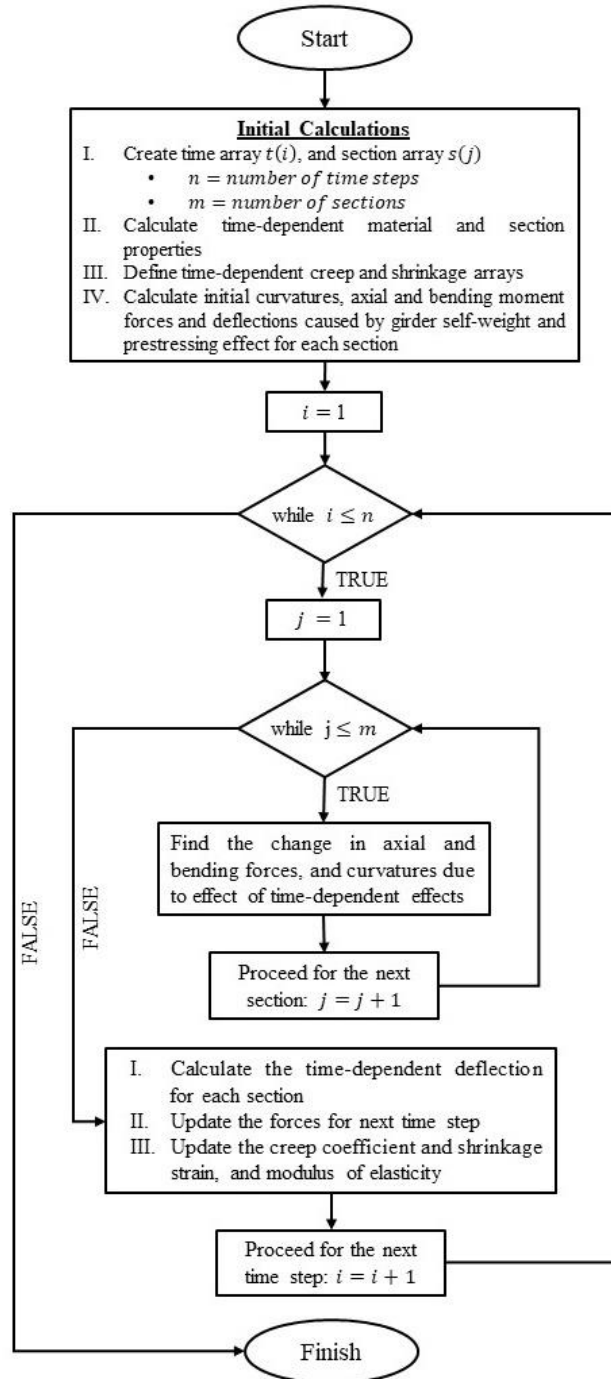


Figure 3.16. Flowchart for Approach No. 5

3.5.6. Approach No. 6

In Approach No. 6, the algorithm used in Approach No. 5 was further improved to allow the use of multiple creep curves rather than a single creep curve. This approach is based on the principle of superposition, multiple creep curves, a time-dependent function for the modulus of elasticity, and multiple time substeps. The method included in this approach removes some of the limitations included in the previous approaches and provides the user the ability to obtain the complete deformation history of the beam including non-composite and composite states as well as deck replacement activities. The proposed prediction methodology relies on the selection of several empirical models which are summarized in Figure 3.17 discussed in the subsequent sections.

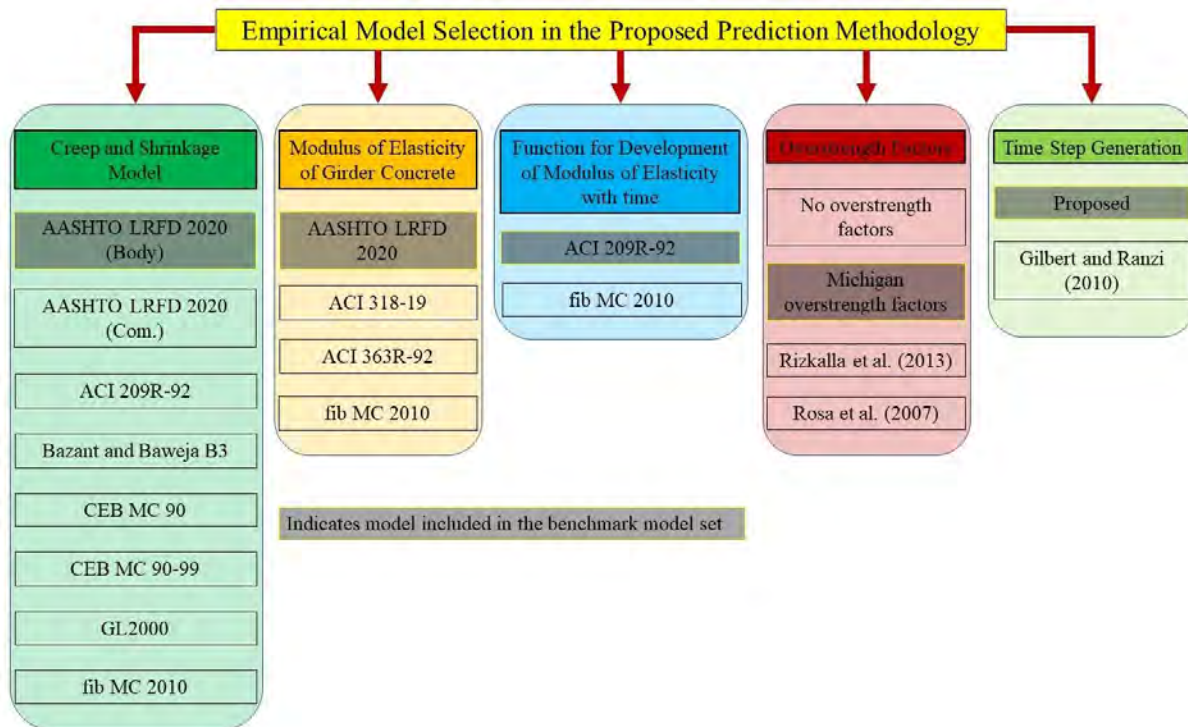


Figure 3.17. Empirical model selection in the proposed prediction methodology

This approach, which is ultimately used in this thesis, has the similar features with previous approach but employed different approach for the estimation of time-dependent deflections. In

previous approaches, Approaches No. 3, 4 and 5, only one creep curve was used for beam and deck concrete to predict time-dependent camber and displacements, and for each time substeps, forces were updated, and remaining creep coefficient and shrinkage strain were used in the calculations. The creep curve for beam concrete was developed based on a loading time of 1 day (prestress release). For the deck concrete, a single creep curve was used assuming that the loading time was the time that corresponded with the placement of the deck. On the other hand, in Approach No. 6, separate creep curves were assigned for each change in force, calculated at the end of each time substeps. While doing so, the forces were not updated because the effect of each force in the force history was considered separately with their own creep curves. For the future time-dependent camber and displacement predictions, remaining creep coefficients for each of these forces were considered. Furthermore, the time for the application of barrier and overlay was separated such that first the barriers are placed, and after some time (if it is desired), the overlay topping is placed on the composite structure. By doing so, another time-dependent calculation can be conducted between application of barrier and overlay load.

With this additional time step, there are 16 and 8 time steps are considered for prestressed concrete girders with and without deck placement event, respectively. These time steps including deck replacement event are as follows:

- 1) detensioning of the strands,
- 2) time from detensioning of the strands to deck placement,
- 3) time at which the deck is placed,
- 4) time from deck placement to placement of barriers (assumed to be 7 days in the analyses),
- 5) time at which the barriers are placed,
- 6) time from the placement of barriers to application of overlay load,

- 7) time at which the overlay load is applied (if desired),
- 8) time from application of overlay load to deck removal for projects that features a deck replacement (assumed to be 50 years in the analyses),
- 9) time at which the old deck together with barrier and overlay is removed,
- 10) time from deck removal to new deck placement (assumed to be 10 days in the analyses),
- 11) time at which the new deck is placed,
- 12) time from new deck placement to placement of new barriers (assumed to be 7 days in the analyses),
- 13) time at which the new barriers are placed,
- 14) time from the placement of new barriers to application of overlay load,
- 15) time at which the overlay load is applied (if desired),
- 16) time from application of overlay load to the end of service life, which is currently considered to be 75 years.

For the analysis, the time at barrier and overlay placements is assumed to be same, which is 7 days after deck is placed. However, the algorithm provides user to have an option to apply overlay load sometime after the bridge is opened to traffic. Meanwhile, when analysis for steel girder is conducted, the first two time steps describe above are eliminated.

Previous approaches do not fully take into account the ageing effect of concrete, which requires the utilization of a separate creep curves. For example, consider a concrete cylinder that was initially loaded at time t_0 (Figure 3.18a) and a new additional force was applied at time t_m . If a single creep curve is used (i.e. the one with a loading time of t_0), the remaining creep coefficient between time t_m and t_n would be small (φ_3). However, if a new creep curve is developed based on a loading time of t_m , then the creep coefficient at time t_n would be greater than the remaining

creep coefficient calculated using the single creep curve ($\varphi_4 \gg \varphi_3$). This constitutes a fundamental difference between Approach No. 5 and Approach No. 6.



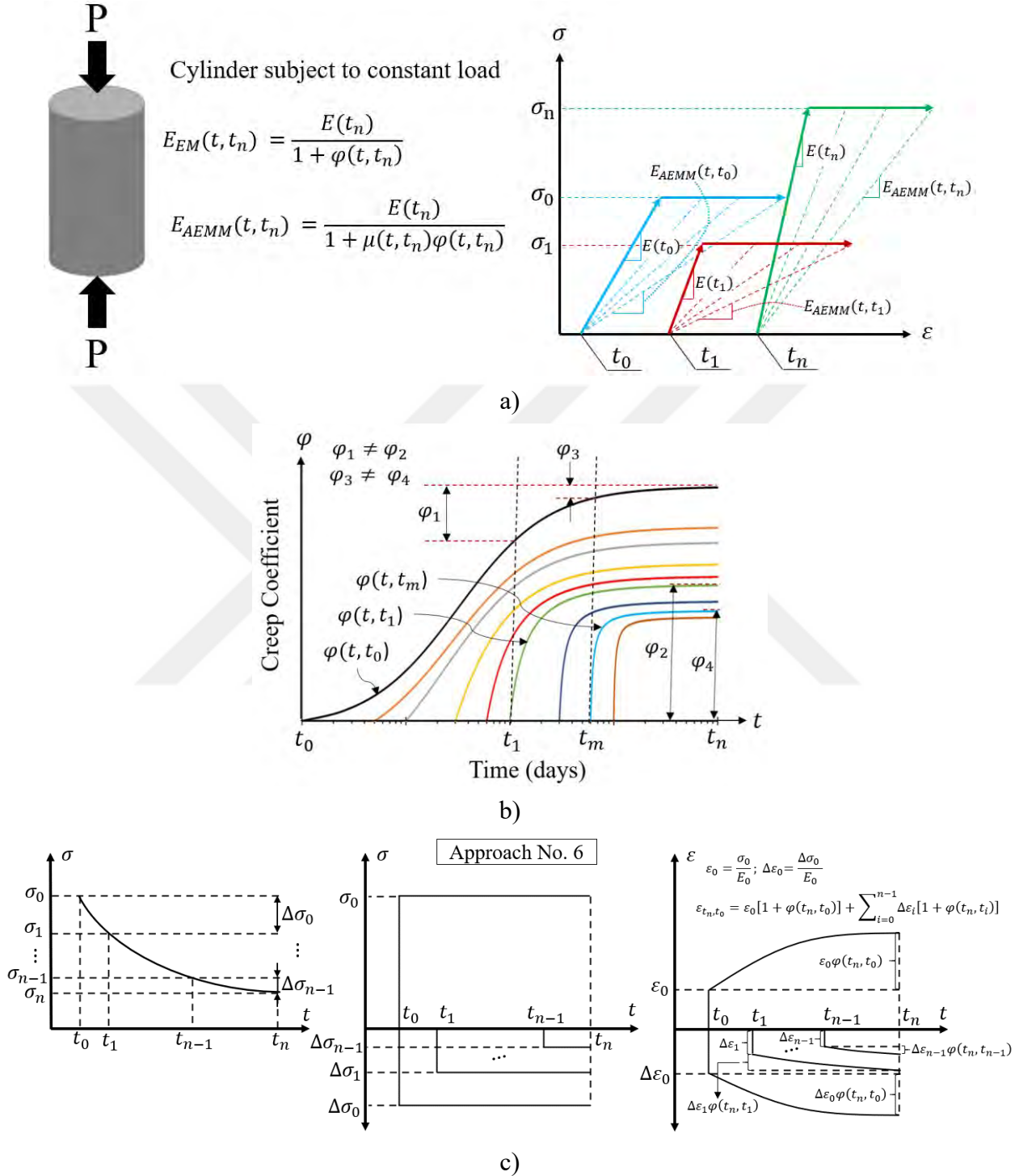


Figure 3.18. a) Illustration of effective modulus (EM) and age adjusted effective modulus (AAEM) concept, b) effect of loading age on creep coefficient according to AASHTO LRFD model for an interior prestressed concrete beam used in the S11 project, c) assumed stress history, idealized stress history, and creep induced strain due to a varying stress history (multiple time steps with multiple creep curves)

Additionally, in Approach No. 6, since each newly created force is associated with its own creep curve, forces at the end of each time step are not updated so that a single axial force and bending moment can be used for the next time step to calculate the corresponding time-dependent effects. Rather, each existing and newly developed force is considered independently, and the principle of superposition is used to calculate total curvatures and axial strain at a given time. The curvature diagram is then numerically integrated to obtain camber and displacements at any location along the span. The only exception is for a deck replacement event, during which all internal axial forces and bending moments present in the deck are summed and are applied as equal and opposite forces to the transformed prestressed concrete girder (i.e. beam and strands) or steel girder when the deck is removed. After this point, any newly created axial forces and bending moment in the beam or in the new deck are associated with separate creep curves. The consideration of multiple sets of axial forces and bending moment in the beam and deck concrete rather than a single set for each component, constitutes another fundamental difference between Approach No. 5 and Approach No. 6. In Approach No. 5 axial forces and bending moments in the beam and deck were summed at the end of each time step so that time-dependent effects for the next time step could be calculated based on single creep curves for beam and deck concrete. In Approach No. 6, this summation is not conducted intentionally to distinguish between the loading time of various events and to facilitate the use of different creep curves.

Another improvement that was implemented in Approach No. 6 is the utilization of creep magnification factors which enables user to take the effect of flexural and tensile creep into account when conducting time-dependent analysis. According to some researchers (Chu and Carreira 1986, Bazant and Oh 1984, Gilbert and Ranzi 2010), the creep of concrete that subjects to uniaxial tension is greater than one that subjects to uniaxial compression. In addition, they stated that to

produce equivalent coefficient for tensile creep, the creep coefficient for concrete under compression may be multiplied by factors 1 to 3. Moreover, to capture the effect of flexural creep of members, since a part of the member subjects to compression, and another part subjects to tension, a coefficient in between compressive creep and tensile creep may be assumed. For instance, if the compressive creep is ϕ , and the tensile creep factor is assumed to be 2, which implies that the tensile creep coefficient is 2ϕ , the flexural creep magnification factor may be assumed in between 1 and 2. It should be noted that while most of the fully prestressed concrete girders are under compression throughout their service lives, the change in axial forces are generally tensile forces. Since each force in the force history matrix is superpositioned by considering corresponding creep curves, their creep induced effects may be better captured by utilizing creep magnification factors.

Another benefit of using creep magnification factors is to capture the effect of creep recovery more accurately. In most of the prestressed concrete girders, the initial stresses on the member are prone to decrease with respect to time due to prestress losses. According to Gilbert and Ranzi (2010), for decreasing stress histories, the method of superposition for capturing the creep recovery is overestimated. Adjusting the creep coefficient for the forces in the force history array may counterbalance the overestimation of creep recovery. Since there is lack of consensus what those creep magnification factors are, their values are assumed to be 1.0 in the analysis.

A detailed implementation of the prediction methodology for the time-dependent camber and displacements is described step by step *below* based on the flowchart provided in Figure 3.19. It should be noted that because in each interval, concrete creeps and shrinks, and steel relaxes, the procedure summarized below should be implemented for each interval described *above* to determine the effect of them on time-dependent deflection. Moreover, Figure 3.19 presents a

flowchart for this approach. Unlike the one provided for Approach No. 5 (Figure 3.16), flowchart given for Approach No. 6 (Figure 3.19) is more descriptive and has more enhanced algorithm.

The explanations of the subscripts mentioned below are as follows:

- m is for the number of loading events,
- n is the number of time steps: $n = m + 1$,
- t_r is the time at which the deck is placed,
- $r - 1$ is the number of loading events until deck placement,
- $m - r$ is the number of loading events between deck placement and final time
- u is the number of sections,
- i is index for the time,
- q is index for section,
- j is an index for the forces in the force history matrix,

1. Time array, t_i , and section array, s_q , are defined.

The time array is created such that the number of time steps increases in a logarithmic fashion to account for the fact that the majority of creep takes places during the early stages of a loading event. The approach used for the creation of time array for the prestressed concrete girders without deck placement event is presented in Table 3.3. An alternative approach is proposed by Gilbert and Ranzi (2010), which is based on a geometric series. The proposed algorithm allows the user to select either approach. The impact of this decision is presented in section 8.6.5.

The section array is created by dividing half of the beam length in an equal number of sections. The user has the option to select how many sections are considered. To obtain a smooth deflected shape for the beam a minimum of 51 sections are recommended. The

influence of the number of sections in the accuracy of camber and displacement predictions is discussed in section 8.6.6.

Table 3.3. Approach used for creation of time array (for the cases with no deck replacement event)

Activity	Time steps between activities	Time step length (days)
Release	Discrete	-
Time between release and deck placement	1 days to 10 days	1
	11 days to 30 days	2
	30 days to time at deck placement	3
Deck Placement	Discrete	-
Time between deck placement and placement of barriers (Typically short – around 7 days)	1 days to 10 days	1
	11 days to 30 days	2
	31 days to 100 days	3
	101 days to 1,000 days	100
	1,001 days to placement of barriers	1,000
Placement of barriers	Discrete	-
Time between placement of barriers and application of overlay load	1 days to 10 days	1
	11 days to 30 days	2
	31 days to 100 days	3
	101 days to 1,000 days	100
	1,001 days to application of overlay	1,000
Application of overlay load	Discrete	-
Time between application of overlay loads and final time (75 years)	1 days to 10 days	1
	11 days to 30 days	2
	31 days to 100 days	3
	101 days to 1,000 days	100
	1,001 days to 10,000 days	1,000
	10,001 days to final time (75 years)	10,000

2. Material and section properties are defined.

Material and section properties for both girder and deck such as modulus of elasticity, modular ratio, transformed moment of inertia, transformed cross-sectional area, composite moment of inertia, composite cross-sectional area, prestressing strand area and eccentricity for each time and section, if applicable, are defined. Please note that some of the parameters are only time-dependent (ex: E_{G_t} , E_{D_t}), some of them are only section-dependent (ex: e_q , A_{ps_q}), and some of them are both time and section-dependent (ex: $I_{ts_{i,q}}$, $A_{ts_{i,q}}$, $I_{c_{i,q}}$, $A_{c_{i,q}}$).

3. Elastic shortening loss is calculated.

Section-dependent elastic shortening loss, Δf_{pESq} , based on AASHTO LRFD Bridge Design Specifications (2020) (C5.9.3.2.3.a-1) is calculated (Eq. 3-72). Note that this is only used for the calculation of relaxation loss (and calculation of effective prestressing stress along the time which is not mentioned in this flowchart).

$$\Delta f_{pESq} = \frac{A_{psq} f_{pbt} (I_G + e_q^2 A_G) - e_q M_{Gswq} A_G}{A_{psq} (I_G + e_q^2 A_G) + \frac{A_G I_G E_{G0}}{E_{ps}}} \quad 3-72$$

A_{psq} = section-dependent strand area; f_{pbt} = stress in the prestressing steel immediately prior to transfer (taken as 202.5 ksi); I_G = moment of inertia of the gross concrete section; e_q = section-dependent eccentricity with respect to centroid of the gross cross-section; A_G = gross cross-sectional area; M_{Gswq} = moment due to girder self-weight along the section; E_{G0} = modulus of elasticity of concrete at transfer; E_{ps} = modulus of elasticity of prestressing steel

4. *Functions for creep coefficients and shrinkage strains are defined.*

Creep and shrinkage functions for girder and deck are defined. Note that since multiple creep curves are generated, time at loading should be provided as an input. $\varphi_{Gi}(t, t_i)$ means that creep coefficient of girder at time t due to load applied at t_i . Subscript “i” in φ_{Gi} defines for different creep curve for different loading times, t_i . Same thing applies for deck as well. While subscript “G” is used for girder, “D” is used for deck. It should be noted that the creep and shrinkage functions for the deck should be defined by considering the global time inputs. For example, say deck is cast 30 days after girder was cast, and the creep coefficient for deck at day 50 due to load applied at 40 days is requested. This is represented as $\varphi_D(50, 40)$ in the algorithm. However, “50” is not the age of deck, but the age of girder. Therefore, while calculating this, the time when the deck is cast should be deducted from input values: $\varphi_D(50 - 30, 40 - 30) = \varphi_D(20, 10)$. Similarly, all time-dependent parameters related to deck such as modulus of elasticity, moment of inertia, and other cross-sectional parameters should

be adjusted for global time inputs. This was accounted for in the algorithm but not explained in the flowchart shown in Figure 3.19.

5. Initial forces that are bending moments and axial forces on the members are calculated.

Initial forces i.e. bending moment and axial force, on girder are calculated: $N_{G_{0,q}}$ and $M_{G_{0,q}}$.

In addition, initial curvature is calculated using “transformed” section properties: $\Sigma_{X_{0,q}}$.

Subscript “0” stands for “release” and “q” is for the sections.

6. Parameter for relaxation loss is defined.

Since remaining relaxation loss is calculated for each time step, initial relaxation loss should be defined. Here, it is assumed to be zero: $f_{pR_{0,q}} = 0$.

7. Stress history for girder is obtained.

Since “stress history concept” is utilized in the Approach No. 6, the stress history for the girder should be obtained in advance. To do so, time-dependent calculations between prestress release and pre-erection time should be performed, and cumulative curvature and force history matrices for girder should be obtained.

8. Force history matrices for the deck are defined.

To recall them inside the loops, force history matrices for the deck (for axial and bending) should be generated: $N_{D_{i,q}}$ and $M_{D_{i,q}}$.

9. Effect of superimposed dead loads on girder and deck is calculated.

The effects of deck load and superimposed dead loads on girder and deck are calculated by considering relative stiffnesses, and the force history matrices i.e. axial and bending for both girder and deck are updated. In the algorithm, the superimposed dead loads are applied after

some time the deck is cast, however, in the flowchart, it is assumed that they are applied simultaneously with the deck placement event to emphasize that the algorithm can consider the effects of superimposed dead loads by utilizing corresponding creep curves.

10. The major loop for time steps is defined.

A loop is defined for the time steps between deck placement and considered final time. It is assumed that there are a total of “n” time steps and “m” loading events. Between prestressing release and deck placement, there are “r” time steps and “r-1” loading events, and between deck placement and final time, there are “m-r+1” loading events and “n-r” time steps. This loop turns for each time step between deck placement and final time defined in step 1.

11. A loop is defined for the number sections considered along the span.

An inner loop is defined for the section considered along the span. It is assumed that there are “u” sections along the span. This loop turns for each section defined in step 1.

12. An index “j” is defined and force history matrices for both girder and deck are assigned to 0.

While utilizing the multiple creep curves for each force in the force history matrices, an index “j” is defined and assigned to 0. Besides, summation terms which are used to represent the cumulative effect of each force in the force history matrix are assigned to 0.

13. Cumulative effect of forces in force history matrices for both girder and deck are calculated

Effect of forces (i.e. axial and bending) in the force history matrix for both girder and deck is considered by using corresponding creep curves and modulus of elasticity values. For this, another loop is constructed to consider all forces in the force history matrices. Since the deck is not cast until time t_r , there can not be any forces on the deck before that time. For this, an if statement is provided. While calculating the cumulative effect, tensile and flexural creep

factors, α and β , should be considered. While the flexural creep factor always magnifies the creep coefficient, tensile creep magnification factor is only accounted for when the considered force is a tensile force, not compression. Another if statement for both deck and girder is provided to quantify this.

14. Tensile creep factors for both girder and deck are assigned to 1, and a stopper is defined for statement check later in the flow.

Assigning tensile creep magnification factors to 1 means that the calculated change in forces for deck and girder for the next time step are assumed to be compressive forces. This assumption will be checked later in step 16, and if not valid, actual tensile creep magnification factors are assigned for the forces which comes out to be positive.

15. Unknown variables are calculated for the next time step.

The change in strains, curvature, axial and bending forces due to time-dependent effects are calculated by using the equations given in Eqns. between 3-88 and 3-96. There are 9 equations and 9 unknowns: $\Delta\epsilon_{G,i,q}$, $\Delta\epsilon_{D,i,q}$, $\Delta N_{G,i,q}$, $\Delta N_{D,i,q}$, $\Delta M_{G,i,q}$, $\Delta M_{D,i,q}$, $\Delta X_{i,q}$, $\Delta\epsilon_{ps,i,q}$, $\Delta N_{ps,i,q}$

16. Calculated change in forces for girder and deck are checked if they are positive.

If either change in axial force on girder or deck comes out to be positive, it means that they are tensile forces, therefore, the creep magnification factors for them should not be 1. Hence, the creep magnification factors for the tensile forces are modified, and stopper variable is set to 0, and step 15 is recalculated. If change in axial forces for the deck and girder are not positive, no action is required.

17. Remaining relaxation loss is calculated.

Total relaxation loss is calculated at considered time, and by subtracting it from the previous time step, remaining relaxation loss is calculated. It should be noted that the relaxation loss depends on total creep and shrinkage loss occurred until considered time. According to AASHTO LRFD Bridge Design Specifications (2020), if the stress in the strands immediately after transfer is smaller than 55% of the yield stress of the strands, no relaxation loss is anticipated. This condition is checked in the flowchart. This condition may be satisfied in the transfer length region where the strands stress is not fully developed.

$$f_{pR_{i+1,q}} = \frac{f_{pt_q}}{K'_L} \frac{\log(24t_{i+1} + 1)}{\log(24t_o + 1)} \left(\frac{f_{pt_q}}{f_{py}} - 0.55 \right) \left[1 - \frac{3 \sum_{v=0}^i \Delta N_{ps_{v,q}}}{f_{pt_q}} \right] K_{i+1,q} \quad 3-73$$

$$\Delta f_{pR_{i,q}} = f_{pR_{i+1,q}} - f_{pR_{i,q}} \quad 3-74$$

$$f_{pt_q} = f_{pj} - \Delta f_{pES_q} \quad 3-75$$

$$K_{i+1,q} = \frac{1}{1 + \frac{E_{ps}}{E_{G_0}} \frac{A_{ps_q}}{A_{C_{i+1,q}}} \left[1 + \frac{A_{C_{i+1,q}} (e_{C_{i+1,q}})^2}{I_{C_{i+1,q}}} \right] [1 + \varphi_{G_0}(t_{i+1}, t_o)]} \quad 3-76$$

f_{pt_q} = section-dependent stress in the prestressing strands immediately after transfer; K'_L = factor accounting type of steel, equal to 45 for low-relaxation steel; t_o = time at release; f_{py} = yield strength of prestressing strands, taken as $0.90f_{pu}$ where f_{pu} is the ultimate tensile strength of prestressing strands; $\sum_{v=0}^i \Delta N_{ps_{v,q}}$ = total creep and shrinkage loss; K = transformed section coefficient (time and section-dependent); f_{pj} = jacking stress of restressing strands, taken $0.75f_{pu}$; Δf_{pES_q} = section-dependent elastic shortening loss; E_{ps} = modulus of elasticity of prestressing strands; E_{G_0} = modulus of elasticity of concrete at release; A_{ps_q} = section-dependent prestressing strand area; A_C = composite cross-sectional area (time and section-dependent); e_C = eccentricity of prestressing strand with respect to centroid of composite section (section-dependent); I_C = composite moment of inertia of the girder (time and section-dependent); $\varphi_{G_0}(t_{i+1}, t_o)$ = creep coefficient at considered time due to loading applied at release.

18. The change in forces on the girder and deck are stored in the force history matrices and cumulative curvature is calculated.

The change in forces on girder and deck (i.e. axial and bending) are added to force history matrix by considering them as separate forces. While doing this, the effect of relaxation loss on girder and deck should be considered. It should be noted that the relaxation loss affects the composite section, thus the effect of relaxation loss on girder and deck should be calculated by using the equations below:

$$\Delta F_{pR_{i,q}} = \Delta f_{pR_{i,q}} A_{ps_q} \text{ \& } \Delta M_{pR_{i,q}} = \Delta F_{pR_{i,q}} e_{c_{i,q}} \quad 3-77$$

$$\Delta N_{Grel_{i,q}} = \frac{\Delta F_{pR_{i,q}} A_G}{A_{C_{i,q}}} + \frac{\Delta M_{pR_{i,q}} A_G (a_{G_{i,q}})}{I_{C_{i,q}}} \quad 3-78$$

$$\Delta N_{Drel_{i,q}} = \frac{\Delta F_{pR_{i,q}} A_{Dt_i}}{A_{C_{i,q}}} - \frac{\Delta M_{pR_{i,q}} A_{Dt_i} a_{D_{i,q}}}{I_{C_{i,q}}} \quad 3-79$$

$$\Delta M_{Grel_{i,q}} = \frac{\Delta M_{pR_{i,q}} I_G}{I_{C_{i,q}}} \text{ \& } \Delta M_{Drel_{i,q}} = \frac{\Delta M_{pR_{i,q}} I_{Dt_i}}{I_{C_{i,q}}} \quad 3-80$$

$\Delta f_{pR_{i,q}}$ = change in stress in strands due to relaxation loss (ksi), A_{ps_q} = strand area ($in.^2$), $\Delta F_{pR_{i,q}}$ = change in force on the strands due to relaxation loss (kips), $e_{c_{i,q}}$ = eccentricity of prestressing strand with respect to centroid of the composite section (in.), $\Delta M_{pR_{i,q}}$ = change in moment on the composite structure due to relaxation loss (kips-in.), A_G = girder area ($in.^2$), $A_{C_{i,q}}$ = area of composite structure ($in.^2$), $a_{G_{i,q}}$ = distance between centroid of girder and centroid of composite section (in.), $I_{C_{i,q}}$ = transformed composite moment of inertia ($in.^4$), $\Delta N_{Grel_{i,q}}$ = change in force on the girder due to relaxation loss (kips), A_{Dt_i} = area of the deck ($in.^2$), $a_{D_{i,q}}$ = distance between centroid of the deck and centroid of the composite section (in.), $\Delta N_{Drel_{i,q}}$ = change in force on the deck due to relaxation loss (kips), I_G = moment of inertia of the girder ($in.^4$), I_{Dt_i} = moment of inertia of the deck ($in.^4$), $\Delta M_{Grel_{i,q}}$ = change in moment on the girder due to relaxation loss (kips-in.), $\Delta M_{Drel_{i,q}}$ = change in moment on the deck due to relaxation loss (kips-in.).

Eqn. 3-77 is for converting the relaxation stress into relaxation force and calculation of change in moment on composite structure due to relaxation loss. Eqns. 3-78 and 3-79 are for the change in axial force on girder and deck due to relaxation loss, respectively. Eqn. 3-80 is for the change in bending moment on girder and deck due to relaxation loss, respectively. Then

net curvature can be calculated by summing up the change in curvature value from the previous net curvature value at considered section ($\Sigma_{X_{i+1,q}}$).

19. Repeat steps between 11 and 18 for each section defined in step 1.

20. The net curvature along the span is obtained.

After each section is covered, net camber/displacement at each section using net curvature values along the span (Σ_X) and second moment-area theorem together with numerical integration is calculated: $\Delta_{i+1,q}$.

21. Repeat steps between 10 and 20 for each time step defined in step 1.

22. Camber and displacement history is obtained.

Once each time step is covered, camber and displacement history can be obtained using the calculated camber/displacement values in step 20 ($\Delta_{i+1,q}$).

The mathematical framework for Approach No. 6 is provided below for different cross-sectional analyses.

Fundamental Equation

$$\varepsilon_{(t_n, t_o)} = \varepsilon_o[1 + \varphi_o(t_n, t_o)] + \sum_{i=0}^m \Delta\varepsilon_i[1 + \varphi_i(t_n, t_i)] \quad 3-81$$

$$\varepsilon_i = \frac{\sigma_i}{E_i}; \Delta\varepsilon_i = \frac{\Delta\sigma_i}{E_i}; n = \text{number of time steps}; m = \text{number of loading events};$$

$$n = m + 1$$

Cross-sectional Analysis for Non-composite (i.e. Transformed) Prestressed Concrete Beams

$$\Delta \varepsilon_{G(t_n, t_o)} = \sum_{i=0}^m \left[\left[\sum_{j=0}^{i-1} \frac{N_{G_j} \alpha_1 [\varphi_{G_j}(t_{i+1}, t_j) - \varphi_{G_j}(t_i, t_j)]}{E_{G_j} A_G} \right] \right. \\ \left. + \frac{\Delta N_{G_i} [1 + \alpha_2 \varphi_{G_i}(t_{i+1}, t_i)]}{E_{G_i} A_G} + [\varepsilon_{ShG}(t_{i+1}) - \varepsilon_{ShG}(t_i)] \right] \quad 3-82$$

$$\text{Where: } N_{G_0} = N_{G_{direct}}^0 + N_G^0, N_{G_{direct}}^0 = 0, N_G^0 = \frac{N_{ts}^0 A_G}{A_{ts_0}} - \frac{M_{ts}^0 a_{ts_0} A_G}{I_{ts_0}};$$

$i > 0 \rightarrow N_{G_i} = \Delta N_{G_i}$ & $N_{G_0} = N_{G_0} + \Delta N_{G_0}$; if $N_{G_i} > 0 \rightarrow \alpha_1 = \text{Tensile creep factor}$,

otherwise $\rightarrow \alpha_1 = 1$; if $\Delta N_{G_i} > 0 \rightarrow \alpha_2 = \text{Tensile creep factor}$, otherwise $\rightarrow \alpha_2 = 1$

$$\Delta X_{(t_n, t_o)} = \sum_{i=0}^m \left[\left[\sum_{j=0}^{i-1} \frac{M_{G_j} \beta [\varphi_{G_j}(t_{i+1}, t_j) - \varphi_{G_j}(t_i, t_j)]}{E_{G_j} I_G} \right] \right. \\ \left. + \frac{\Delta M_{G_i} [1 + \beta \varphi_{G_i}(t_{i+1}, t_i)]}{E_{G_i} I_G} \right] \quad 3-83$$

$$\text{Where: } M_{G_0} = M_{G_{direct}}^0 + M_G^0, M_{G_{direct}}^0 = 0, M_G^0 = \frac{M_{ts}^0 I_G}{I_{ts_0}};$$

$$i > 0 \rightarrow M_{G_i} = \Delta M_{G_i} \text{ \& } M_{G_0} = M_{G_0} + \Delta M_{G_0}$$

$$\underline{\text{Note:}} \quad N_{ts}^0 = -P_e, M_{ts}^0 = M_{G_{sw}} - P_e e_{tr}$$

$$\Delta \varepsilon_{ps(t_n, t_o)} = \frac{\sum_{i=0}^m \Delta N_{ps_i}}{E_{ps} A_{ps}} \quad 3-84$$

$$\Delta N_{G_i} + \Delta N_{ps_i} = 0 \quad 3-85$$

$$\Delta M_{G_i} + \Delta N_{ps_i} e = 0 \quad 3-86$$

$$\Delta \varepsilon_{ps(t_n, t_o)} = \Delta \varepsilon_{G(t_n, t_o)} + \Delta X_{(t_n, t_o)} e \quad 3-87$$

Cross-sectional Analysis for Composite Prestressed Concrete Beams

$$\Delta \varepsilon_{G(t_n, t_d)} = \sum_{i=r}^m \left[\left[\sum_{j=0}^{i-1} \frac{N_{G_j} \alpha_1 [\varphi_{G_j}(t_{i+1}, t_j) - \varphi_{G_j}(t_i, t_j)]}{E_{G_j} A_G} \right] \right. \\ \left. + \frac{\Delta N_{G_i} [1 + \alpha_2 \varphi_{G_i}(t_{i+1}, t_i)]}{E_{G_i} A_G} + [\varepsilon_{ShG}(t_{i+1}) - \varepsilon_{ShG}(t_i)] \right] \quad 3-88$$

$$\text{Where: } N_{G_0} = N_{G_{direct}}^0 + N_G^0, N_{G_{direct}}^0 = 0, N_G^0 = \frac{N_{ts}^0 A_G}{A_{ts_0}} - \frac{M_{ts}^0 a_{ts_0} A_G}{I_{ts_0}},$$

$$N_{G_i} = \Delta N_{G_i} \text{ if } 0 < i < (r-1), N_{G_{r-1}} = \Delta N_{G_{r-1}} + \frac{N_c^0 A_G}{A_{c_0}} + \frac{M_c^0 a_{G_0} A_G}{I_{c_0}}, N_{G_i} = \Delta N_{G_i} \text{ if } i \geq r,$$

if $N_{G_i} > 0 \rightarrow \alpha_1 = \text{Tensile creep factor}$, otherwise $\rightarrow \alpha_1 = 1$; if $\Delta N_{G_i} > 0 \rightarrow \alpha_2 = \text{Tensile creep factor}$, otherwise $\rightarrow \alpha_2 = 1$;

$$\Delta \varepsilon_{D(t_n, t_d)} = \sum_{i=r}^m \left[\left[\sum_{j=r}^{i-1} \frac{N_{D_j} \alpha_1 [\varphi_{D_j}(t_{i+1}, t_j) - \varphi_{D_j}(t_i, t_j)]}{E_{D_j} A_D} \right] \right. \\ \left. + \frac{\Delta N_{D_i} [1 + \alpha_2 \varphi_{D_i}(t_{i+1}, t_i)]}{E_{D_i} A_D} + [\varepsilon_{ShD}(t_{i+1}) - \varepsilon_{ShD}(t_i)] \right] \quad 3-89$$

$$\text{Where: } N_{D_r} = N_{D_{direct}}^0 + N_D^0, N_{D_{direct}}^0 = 0, N_D^0 = \frac{N_c^0 n_0 A_D}{A_{c_0}} - \frac{M_c^0 a_{D_0} n_0 A_D}{I_{c_0}},$$

$$i > r \rightarrow N_{D_i} = \Delta N_{D_i} \text{ \& } N_{D_r} = N_{D_r} + \Delta N_{D_r};$$

if $N_{D_i} > 0 \rightarrow \alpha_1 = \text{Tensile creep factor}$, otherwise $\rightarrow \alpha_1 = 1$;

if $\Delta N_{D_i} > 0 \rightarrow \alpha_2 = \text{Tensile creep factor}$, otherwise $\rightarrow \alpha_2 = 1$

$$\Delta X_{(t_n, t_d)} = \sum_{i=r}^m \left[\left[\sum_{j=0}^{i-1} \frac{M_{G_j} \beta [\varphi_{G_j}(t_{i+1}, t_j) - \varphi_{G_j}(t_i, t_j)]}{E_{G_j} I_G} \right] \right. \\ \left. + \frac{\Delta M_{G_i} [1 + \beta \varphi_{G_i}(t_{i+1}, t_i)]}{E_{G_i} I_G} \right] \quad 3-90$$

Where: $M_{G_0} = M_{G_{direct}}^0 + M_G^0, M_{G_{direct}}^0 = 0, M_G^0 = \frac{M_{ts}^0 I_G}{I_{ts0}}$,

$M_{G_i} = \Delta M_{G_i}$ when $0 < i < (r-1), M_{G_{r-1}} = \Delta M_{G_{r-1}} + \frac{M_c^0 I_G}{I_{c0}}, M_{G_i} = \Delta M_{G_i}$ when $i \geq r$

$$\Delta X_{(t_n, t_d)} = \sum_{i=r}^m \left[\left[\sum_{j=r}^{i-1} \frac{M_{D_j} \beta [\varphi_{D_j}(t_{i+1}, t_j) - \varphi_{D_j}(t_i, t_j)]}{E_{D_j} I_D} \right] \right. \\ \left. + \frac{\Delta M_{D_i} [1 + \beta \varphi_{D_i}(t_{i+1}, t_i)]}{E_{D_i} I_D} \right] \quad 3-91$$

Where: $M_{D_0} = M_{D_{direct}}^0 + M_D^0, M_{D_{direct}}^0 = 0, M_D^0 = \frac{M_c^0 n_o I_D}{I_{c0}}$,

$i > r \rightarrow M_{D_i} = \Delta M_{D_i} \text{ \& } M_{D_r} = M_{D_r} + \Delta M_{D_r}$

Note: $N_{ts}^0 = -P_e, M_{ts}^0 = M_{G_{sw}} + M_{dsw} - P_e e_{tr}, N_c^0 = 0, M_c^0 = M_{bsw} + M_{osw}$, and

$\beta = \text{flexural creep factor}$

$$\Delta \varepsilon_{ps(t_n, t_d)} = \frac{\sum_{i=r}^n \Delta N_{ps_i}}{E_{ps} A_{ps}} \quad 3-92$$

$$\Delta N_{D_i} + \Delta N_{G_i} + \Delta N_{ps_i} = 0 \quad 3-93$$

$$\Delta M_{G_i} + \Delta M_{D_i} - \Delta N_{D_i} a + \Delta N_{ps_i} e = 0 \quad 3-94$$

$$\Delta \varepsilon_{D(t_n, t_d)} = \Delta \varepsilon_{G(t_n, t_d)} - \Delta X_{(t_n, t_d)} a \quad 3-95$$

$$\Delta \varepsilon_{ps(t_n, t_d)} = \Delta \varepsilon_{G(t_n, t_d)} + \Delta X_{(t_n, t_d)} e \quad 3-96$$

Cross-sectional Analysis for Composite Steel Beams

$$\Delta \varepsilon_{D(t_n, t_d)} = \sum_{i=r}^m \left[\left[\sum_{j=r}^{i-1} \frac{N_{D_j} \alpha_1 [\varphi_{D_j}(t_{i+1}, t_j) - \varphi_{D_j}(t_i, t_j)]}{E_{D_j} A_D} \right] \right. \\ \left. + \frac{\Delta N_{D_i} [1 + \alpha_2 \varphi_{D_i}(t_{i+1}, t_i)]}{E_{D_i} A_D} + [\varepsilon_{ShD}(t_{i+1}) - \varepsilon_{ShD}(t_i)] \right] \quad 3-97$$

$$\text{Where: } N_{D_r} = N_{D_{direct}}^0 + N_D^0, N_{D_{direct}}^0 = 0, N_D^0 = \frac{N_c^0 n_0 A_D}{A_{c_0}} - \frac{M_c^0 a_{D_0} n_0 A_D}{I_{c_0}};$$

$$i > r \rightarrow N_{D_i} = \Delta N_{D_i} \text{ \& } N_{D_r} = N_{D_r} + \Delta N_{D_r}$$

if $N_{D_i} > 0 \rightarrow \alpha_1 = \text{Tensile creep factor}$, otherwise $\rightarrow \alpha_1 = 1$; if $\Delta N_{D_i} > 0 \rightarrow \alpha_2 = \text{Tensile creep}$

factor, otherwise $\rightarrow \alpha_2 = 1$

$$\Delta X_{(t_n, t_d)} = \sum_{i=r}^m \left[\left[\sum_{j=r}^{i-1} \frac{M_{D_j} \beta [\varphi_{D_j}(t_{i+1}, t_j) - \varphi_{D_j}(t_i, t_j)]}{E_{D_j} I_D} \right] \right. \\ \left. + \frac{\Delta M_{D_i} [1 + \beta \varphi_{D_i}(t_{i+1}, t_i)]}{E_{D_i} I_D} \right] \quad 3-98$$

$$\text{Where: } M_{D_r} = M_{D_{direct}}^0 + M_D^0, M_{D_{direct}}^0 = 0, M_D^0 = \frac{M_c^0 n_0 I_D}{I_{c_0}};$$

$$i > r \rightarrow M_{D_i} = \Delta M_{D_i} \text{ \& } M_{D_r} = M_{D_r} + \Delta M_{D_r}$$

Note: $N_c^0 = 0$, $M_c^0 = M_{bsw} + M_{osw}$, and β = flexural creep factor

$$\Delta X_{(t_n, t_d)} = \frac{\Delta M_G}{E_G I_G} \quad 3-99$$

$$\Delta \varepsilon_{G(t_n, t_d)} = \frac{\Delta N_G}{E_G A_G} \quad 3-100$$

$$\Delta N_{D_i} + \Delta N_{G_i} = 0 \quad 3-101$$

$$\Delta M_{G_i} + \Delta M_{D_i} - \Delta N_{D_i} a = 0 \quad 3-102$$

$$\Delta \varepsilon_{D(t_n, t_d)} = \Delta \varepsilon_{G(t_n, t_d)} - \Delta X_{(t_n, t_d)} a \quad 3-103$$

Where:

$\Delta \varepsilon_{G(t_n, t_0)}$ is the change in strain in the girder between times t_n and t_0 ,

$\Delta \varepsilon_{D(t_n, t_0)}$ is the change in strain in the deck between times t_n and t_0 ,

$\Delta \varepsilon_{ps(t_n, t_0)}$ is the change in strain in the prestressing strands between times t_n and t_0 ,

$\Delta X_{(t_n, t_0)}$ is the change in curvature between times t_n and t_0 ,

N_{G_i} and N_{D_i} are the axial forces on the girder and deck, respectively,

M_{G_i} and M_{D_i} are the moment forces on the girder and deck, respectively,

ΔN_{G_i} , ΔN_{D_i} and ΔN_{ps_i} are the change in axial force on the girder, deck and prestressing strand, respectively,

ΔM_{G_i} and ΔM_{D_i} are the change in bending moment on the girder and deck, respectively,

$N_{Gdirect}^0$ and $N_{Ddirect}^0$ are the axial force directly applied to the girder and deck, respectively,

N_G^0 and N_D^0 are the axial force on the girder and deck caused by forces applied on transformed/composite section, respectively,

$M_{Gdirect}^0$ and $M_{Ddirect}^0$ are the bending moment directly applied to the girder and deck, respectively,

M_G^0 and M_D^0 are the bending moment on the girder and deck caused by forces applied on transformed/composite section, respectively,

N_{ts}^0 and M_{ts}^0 are the axial force and bending moment on the transformed section, respectively,

N_C^0 and M_C^0 are the axial force and bending moment on the composite section, respectively,

M_{Gsw} is the bending moment due to girder self-weight applied to the transformed section (girder and strands),

M_{bsw} and M_{osw} are the moment created by barrier and overlay load on composite section, respectively,

E_{G_i} and E_{D_i} are the moduli of elasticity of girder and deck concrete, respectively,

E_{ps} is the modulus of elasticity of prestressing strands.

I_{ts_0} is the moment of inertia of transformed section (girder and strands) at release,

I_{C_0} is the moment of inertia of composite section when composite action starts,

I_G is the moment of inertia of the girder alone,

I_D is the moment of inertia of the deck alone,

A_{ts_0} is the area of the transformed section (girder and strands) at release,

A_{C_0} is the area of the composite section when composite action initiates,

A_G , A_D and A_{ps} are the area of the girder, deck and prestressing strand, respectively,

e is the eccentricity of prestressing strands with respect to centroid of girder,

a is the distance between centroid of deck and girder,

a_D is the distance between centroid of deck and centroid of composite section,

a_G is the distance between centroid of girder and centroid of composite section,

e_{tr} is the eccentricity of prestressing strands with respect to centroid of transformed section (girder and strands),

a_{ts} is the distance between centroid of the girder and centroid of the transformed (girder and strands) section,

$y_{Gbottom}$ is the distance between centroid of girder with respect to the bottom of girder,

$y_{psbottom}$ is the distance between centroid of prestressing strand group with respect to the bottom of girder,

n_o is the modular ratio for deck,

P_e is the effective force in the prestressing strands,

α_1 and α_2 are tensile creep magnification factors,

β is the flexural creep magnification factor,

$\varphi_{G_i}(t_n, t_i)$ is the creep coefficient of girder at time t_n due to load applied at t_i ,

$\varphi_{D_i}(t_n, t_i)$ is the creep coefficient of deck at time t_n due to load applied at t_i ,

$\varepsilon_{shG}(t_n)$ is the shrinkage of girder at time t_n ,

$\varepsilon_{shD}(t_n)$ is the shrinkage of deck at time t_n .

It should be noted that for the prestressed concrete girders, prestress losses due to relaxation should be added to losses caused by creep and shrinkage to calculate the magnitude of the prestressing force at the end of the considered time step and at the beginning of the next time step as presented in the Figure 3.19. Although the flowchart in Figure 3.19 uses the equations for cross-sectional analysis for composite section (Eqns. 3-88 - 3-96), the same flow can also be implemented for other cross-sectional analysis types i.e. steel beams with cast-in-place deck and non-composite (transformed) prestressed concrete beams.

The final version of the program can be provided upon request (Cakmak et al. 2021). As it is stated above, the user does not need to have MATLAB to run the program but must have the MATLAB Runtime which is downloaded during the installation of the program. An installation guide about how to install the *MDOTCamber* program is presented in APPENDIX H. In addition, APPENDIX D provides the user's manual for the *MDOTCamber* program.

The method described in this section has the following advantages:

- ❖ It accounts for the creep and shrinkage behavior of the beam and deck individually and allows the user to select the desired shrinkage and creep models.
- ❖ Utilizing separate creep curves for the forces with different load application times allow time-dependent deflections to be estimated more accurately.
- ❖ It accurately calculates prestress losses at each time step by considering the adopted shrinkage and creep models and relaxation of prestressing steel.
- ❖ It accounts for the reduction of material stiffness due to creep and increase in material stiffness due to ageing by effective modulus.
- ❖ It enables to draw deformed shape of the girder at each time step.

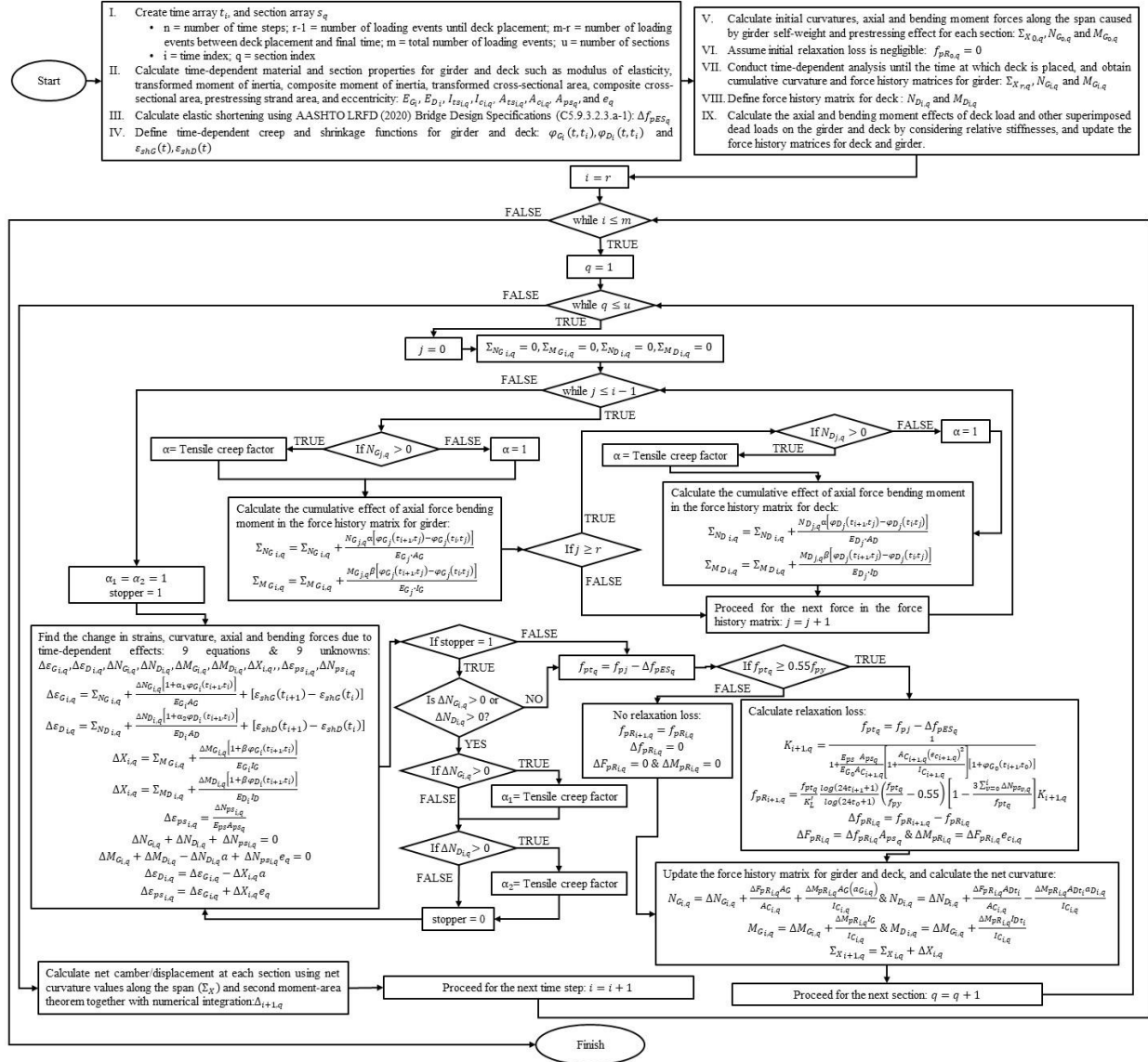


Figure 3.19. Flowchart for Approach No. 6

3.6. Modulus of Elasticity Prediction

Modulus of elasticity is one of the most influential parameters on beam camber and displacement as it represents the stiffness of the material. Its development with time is also important when predicting time dependent camber and deflections.

In this thesis, four models are presented for predicting the modulus of elasticity at release and at 28 days: AASHTO LRFD 2020, ACI 318-19, ACI 363R-92 and fib MC. Additionally, two models for predicting the development of modulus with time are provided: ACI 209R-92 and fib MC 2010. The proposed prediction methodology allows the user to select the desired model. In this part, the formulations for each prediction and development model are presented. A comprehensive discussion about the effect of selected model for prediction of modulus of elasticity on service camber and displacements is presented in sections 5.2.2 and 6.2.2. In addition to this, section 4.6 provides a comparison study between measured and calculated moduli of elasticity using abovementioned modulus of elasticity models.

3.6.1. Models for Modulus of Elasticity Prediction at Release and 28 Days

There are many models to estimate the modulus of elasticity of concrete, including, but not limited to AASHTO LRFD 2020, ACI 318-19, ACI 363R-92 and fib MC 2010 models. In this part, these four prediction models are presented.

3.6.1.1. AASHTO LRFD 2020

In this model, modulus of elasticity is expressed as function of aggregate type, K_1 , concrete unit weight, w_c , and compressive strength of concrete at release, f'_{ci} , or at 28 days, f'_c (Eqns. 3-104 and 3-105). Eqns. 3-104 and 3-105 are intended for normal weight concrete with design compressive strength up to 15.0 ksi and lightweight concrete up to 10.0 ksi, with unit weights

between 0.090 and 0.155 kcf. The units for w_c are kcf and for f'_{ci} , or f'_c are ksi. The modulus of elasticity is computed in ksi.

$$E_{ci} = 120,000K_1w_c^{2.0}(f'_{ci})^{0.33} \quad 3-104$$

$$E_{c28} = 120,000K_1w_c^{2.0}(f'_c)^{0.33} \quad 3-105$$

For normal weight concrete, $w_c = 0.145$ kcf for $f'_c \leq 5$ ksi and, $w_c = 0.140 + 0.001f'_c$ kcf for $5.0 < f'_c \leq 15.0$ ksi

3.6.1.2. ACI 318-19

This model is similar with the AASHTO LRFD 2020 model in format (Eqns. 3-106 and 3-107). However, the powers to which the unit weight of concrete and compressive strength are raised are different. In addition, the ACI 318-19 model does not distinguish between various types of aggregates. The units for w are pcf and for f'_{ci} , or f'_c are psi. The modulus of elasticity is computed in psi.

$$E_{ci} = 33w^{1.5}\sqrt{f'_{ci}} \quad 3-106$$

$$E_{c28} = 33w^{1.5}\sqrt{f'_c} \quad 3-107$$

3.6.1.3. ACI 363R-92

This model is similar to the ACI 318-19 model in terms the parameters that the modulus of elasticity is expressed as a function of (Eqns. 3-108 and 3-109). The units for w_c are pcf and for f'_{ci} , or f'_c are psi. The modulus of elasticity is computed in psi.

$$E_{ci} = \left(40,000\sqrt{f'_{ci}} + 10^6\right)\left(\frac{w_c}{145}\right)^{1.5} \quad 3-108$$

$$E_{c28} = (40,000\sqrt{f'_c} + 10^6) \left(\frac{w_c}{145}\right)^{1.5} \quad 3-109$$

3.6.1.4. fib MC 2010

This model, similar to the AASHTO LRFD 2020 model expresses the modulus of elasticity as a function of concrete compressive strength and aggregate type (Eqns. 3-110 and 3-111). However, unlike the other models it does not include the unit weight of concrete as a parameter. The aggregate type dependent parameter can be selected based on Table 3.4.

$$E_{ci} = 2760\alpha_E \sqrt[3]{f'_{ci}} \quad 3-110$$

$$E_{c28} = 2760\alpha_E \sqrt[3]{f'_c} \quad 3-111$$

Table 3.4. Parameter for aggregate type

Aggregate type	α_E
Basalt, dense limestone	1.2
Quartzite	1.0
Limestone	0.9
Sandstone	0.7

Comparison of Modulus of Elasticity Prediction Models

Consider a normal weight concrete beam with aggregate factor 1.0 and initial and 28 days compressive strengths of 7 and 10 ksi, respectively. The unit weight of concrete for all models except for the AASHTO LRFD 2020 model is assumed as 145 pcf, whereas for the AASHTO LRFD 2020 model, it is computed based on Table 3.5.1-1 in AASHTO LRFD Bridge Design Specifications (2020), which relates compressive strength to unit weight. The modulus of elasticity predictions at release and at 28 days using the four different models discussed above are presented in Table 3.5.

Table 3.5. Comparison of modulus of elasticity prediction models

Modulus of Elasticity	Prediction Models			
	AASHTO LRFD 2020	ACI 318-19	ACI 363R-92	fib MC 2010
E_{ci} (ksi)	5165	4821	4347	5280
E_{c28} (ksi)	5817	5762	5000	5946

As can be seen from the Table 3.5, the fib MC 2010 and AASHTO LRFD 2020 models produce the highest estimates followed by ACI 318-19 model and ACI 363R-92 model.

3.6.2. Prediction of Modulus of Elasticity as a Function of Time

There are additional parameters required to determine the development of modulus of elasticity such as curing type, cement type and aggregate type with time. Using these parameters together with the 28 day modulus, a function for elastic modulus development can be determined. On the other hand, if both initial (at release) and 28 day modulus values are known, then these parameters can be back-calculated such that it enforces the development function to pass from given modulus of elasticity values at initial and 28 days. In this case, a more accurate development function may be obtained. A comprehensive discussion about the effect of selected model of prediction of modulus of elasticity function with respect to time on service camber and displacements is presented in sections 5.2.3 and 6.2.3. In addition to this, section 4.7 provides a comparison study between measured and predicted development of modulus of elasticity with respect to time using abovementioned development models.

3.6.2.1. ACI 209R-92

In this model the development of modulus of elasticity with time is expressed as function of modulus of elasticity at 28 days and two parameters which are selected based on the type of cement used and curing method (Eqn. 3-112). The determination of α and β is based on Table 3.6. This model covers only concretes that features either cement type I or III. In the proposed

prediction methodology, the user has two options to obtain the development of modulus with time. The first include the use of 28 day modulus and parameters α and β . However, in this approach, it is possible that the predicted modulus at release may not match the measured modulus if such information is available or if it can be obtained using one of the four models presented above that express modulus as a function of compressive strength.

$$E_c(t) = \left(\frac{t}{\alpha + \beta t} \right)^{0.5} E_{c28} \quad 3-112$$

Where:

$E_c(t)$ is modulus of elasticity function with time,

E_{c28} is the 28 day modulus of elasticity of concrete,

α and β are the parameters depending on curing type and cement type (Table 3.6).

Table 3.6. Parameter for ACI 209R-92 model

Type of cement	Moist-cured concrete		Steam-cured concrete	
	α	β	α	β
I	4	0.85	1	0.95
III	2.3	0.92	0.7	0.98

Alternatively, the development of modulus with time can be obtained by using the initial modulus and 28 day modulus as anchor points and back calculating the parameters α and β such that predicted modulus at release and 28 day matches the measured or assumed one. This can be achieved using Eqns. 3-113 and 3-114.

$$\beta = \frac{E_{c28}^2 \cdot t_1 - 28E_{ci}^2}{E_{ci}^2 \cdot (t_1 - 28)} \quad 3-113$$

$$\alpha = 28(1 - \beta) \quad 3-114$$

Where:

t_1 is the time when strands are released,

E_{ci} is the modulus of elasticity value when strands are released.

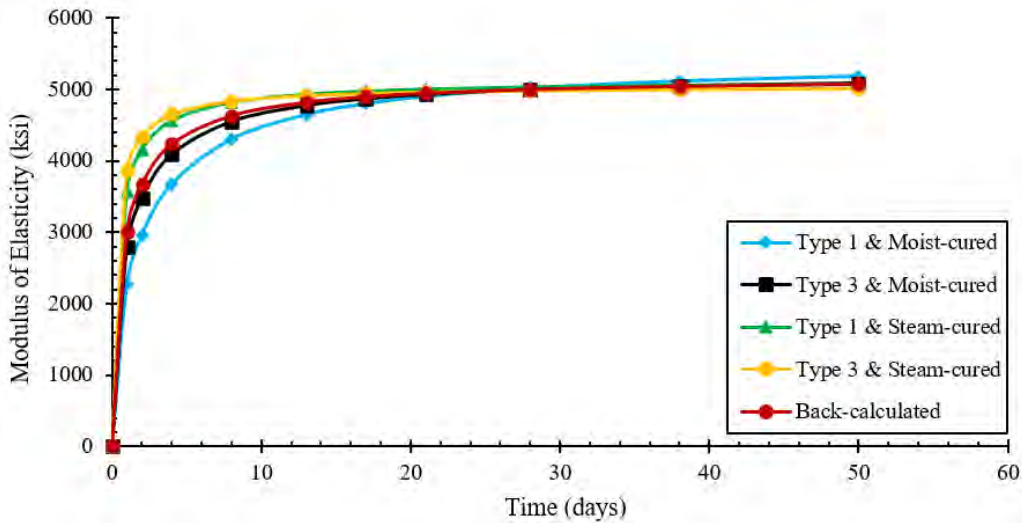


Figure 3.20. Modulus of elasticity as a function of time based on the ACI 209R-92 Model

Figure 3.20 shows the development of modulus of elasticity as a function of time by considering a concrete with an initial (1 day) and 28 day modulus of elasticity of 3000 ksi and 5000 ksi, respectively. Figure 3.20 suggests that the modulus of elasticity reaches a stable value after 28 days with only minor differences between concretes featuring cement type I and III and those that are moist and steam cured. Additionally, even if the parameters α and β were back calculated based on the specified initial and 28 day modulus, the modulus after 28 days is similar with the other approach. It is worth noting, however, that there are some differences between the predicted modulus up to 28 days depending on which approach, which type of cement and which method of curing is selected. According to this model, steam curing helps the stiffening of concrete during the first 28 days compared to moist curing. The back calculated curve is similar to that obtained for Type III cement and moist cured concrete.

3.6.2.2. fib MC 2010

In this model the development of modulus of elasticity with time is expressed as a function modulus of elasticity at 28 days as well as the strength class of cement used (Eqns. 3-115 and 3-116). Unlike the previous model, the type of curing is not a parameter that influences the development of modulus with time. The parameter s that reflects the strength class of cement used in the mix may be obtained based on Table 3.7. The mean compressive strength may be taken as 1.2 ksi more than specified strength.

$$E_c(t) = \beta_E(t) E_{c28} \quad 3-115$$

$$\beta_E(t) = \sqrt{e^{s \left(1 - \sqrt{\frac{28}{t}}\right)}} \quad 3-116$$

Where:

$\beta_E(t)$ is a function to describe the development function,

s is a coefficient depending on the strength class of cement (Table 3.7)

Table 3.7. Coefficient s for strength class of cement based on fib MC 2010 model

Mean compressive strength (ksi)	Strength class of cement	s
≤ 8.7	32.5 N	0.38
	32.5 R, 42.5 N	0.25
	42.5 R, 52.5 N, 52.5 R	0.20
> 8.7	all classes	0.20

Moreover, if both initial and 28 day modulus of elasticity values are known, the parameter “ s ” may be back-calculated using Eqn. 3-117. The proposed prediction methodology allows the user to select either approach within each prediction model for estimating the development of modulus with time.

$$s = \frac{\ln \left[\left(\frac{E_{ci}}{E_{c28}} \right)^2 \right]}{1 - \sqrt{\frac{28}{t_1}}} \quad 3-117$$

Figure 3.21 shows the development of modulus as a function of time using different classes for the strength of cement by considering a concrete with initial (1 day) and 28 day modulus of elasticity of 3000 psi and 5000 psi, respectively. The curve obtained by back calculating the coefficient s is also presented. Figure 3.21 suggests that the development of modulus with time is generally similar for all cases with lower strength cements featuring a lower modulus. This is different from the previous model in which pronounced differences in the development of modulus with time up to 28 days were observed depending on the type of curing method.

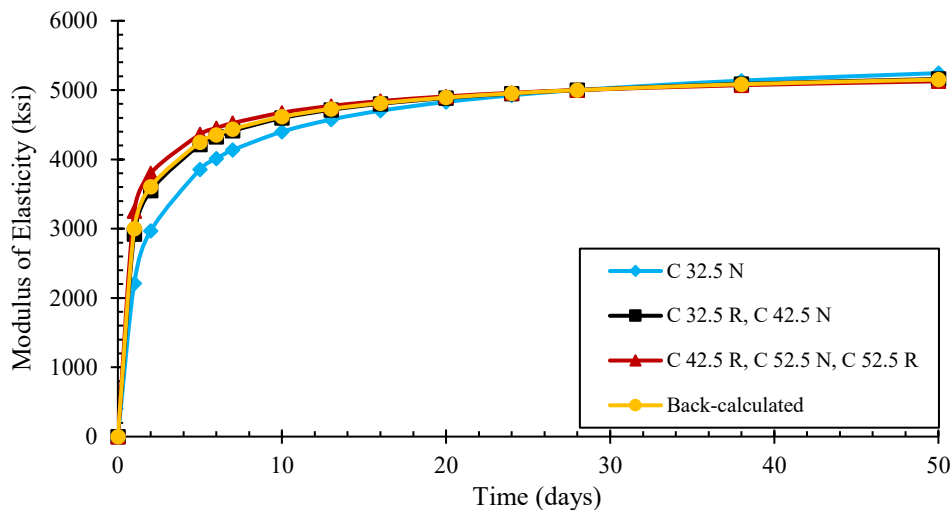


Figure 3.21. Modulus of Elasticity as a function of time based on the fib MC 2010 Model

3.7. Creep and Shrinkage Models

Creep, which is defined as the time-dependent increase of strain in hardened concrete subjected to sustained stress (ACI 209R-92), is one of the most influential parameters while calculating the time-dependent deformation of members. Creep coefficient, defined as the ratio of creep strain to initial elastic strain, is used to predict time-dependent strains by multiplying with

elastic strain due to permanent loads. Therefore, it is important to determine the development of creep coefficient accurately with respect to time to predict the time-dependent deformations of the prestressed concrete accurately.

Moreover, shrinkage, which is defined as volume reduction due to loss of water, is another factor that leads to time-dependent deformations in the members. There are two types of shrinkage: autogenous shrinkage and drying shrinkage. The first is the shrinkage occurring in the absence of moisture exchange due to hydration reaction taking place in cement matrix. The latter, on the other hand, is defined as shrinkage taking place in a specimen that is allowed to dry (ACI 209.2R-08). From these explanations, it can be inferred that even if a specimen is not exposed to environmental effects, there may be some shrinkage occurring due to chemical reactions inside the cement matrix. Some of the models defined in this part takes autogenous shrinkage into account, however when compared to drying shrinkage, its contribution is less to total shrinkage. It is assumed that shrinkage takes places uniformly along the member cross-section, which is not accurate all the time, but it does not lead to significant errors in the calculations. Therefore, unlike creep, shrinkage is not one of the significant factors that affect time-dependent deformation of prestressed concrete members alone. On the other hand, differential shrinkage between deck and girder may be significant. Generally, when the deck is placed on the girder, most of the shrinkage of prestressed concrete girder has taken place. Therefore, the difference in shrinkage between deck and girder may cause significant downward deflection.

The calculation of long-term camber and displacements is highly dependent on the creep and shrinkage models that are selected. ACI 209.2R-08 (2008) provides five models for calculating shrinkage and creep in hardened concrete, namely: ACI 209R-92 model, Bažant-Baweja B3 model, CEB MC90, CEB MC90-99 model and GL2000 model. The Precast Prestressed Concrete

Institute (PCI) bridge design manual (2016) recommends two methods for estimating the creep coefficient and shrinkage strain. The first is the same with the ACI 209.2R-08 (2008) model and the second is based on modifications suggested by Huo (1997). AASHTO LRFD Bridge Design Specifications (2020) also provide models for estimating shrinkage and creep in hardened concrete. LeRoy et al. (1996), Tadros et al. (2003), and Lopez and Khan (2004) developed models for estimating the creep behavior of high strength concrete. The proposed methodology allows the selection of the following creep and shrinkage models: 1) AASHTO LRFD Body 2020 (provisions in the main body), 2) AASHTO LRFD Commentary 2020 (provisions in the commentary), 3) ACI 209R-92, 4) Bažant-Baweja B3 model, 5) CEB MC90, 6) CEB MC90-99, 7) GL2000, and 8) fib MC 2010. APPENDIX C shows the formulae and range of applicability for each of these models in detail.

Figure 3.22 illustrates how the free shrinkage strain and creep coefficient vary with time for a PCBT 54 beam used in the S11 project and the corresponding deck based on different shrinkage and creep models. The variation is shown using a logarithmic timescale for fixed values of initial concrete compressive strength, f_{ci} , ambient relative humidity, h , and volume-to-surface ratio, V/S . Required parameters for the beam and deck are obtained from concrete mixture properties of a beam of S11 project, and a normal weight – fly ash (NW-FA) mix composition properties, respectively, as shown in Table 3.8.

Table 3.8. Concrete mixture properties for girder and deck

Property	Girder	Deck
Water content	250 lb/yd ³	286 lb/yd ³
Cement content	735 lb/yd ³	476 lb/yd ³
Coarse aggregate content	1642 lb/yd ³	–
Fine aggregate content	1186 lb/yd ³	–
Total aggregate content	2828 lb/yd ³	2814 lb/yd ³

The variation of creep coefficient with time generally exhibits an asymptotic behavior. As can be seen, the creep coefficient at a given beam age can vary significantly depending on which model is chosen. The same conclusion can be drawn for beam and deck shrinkage. The considered models include the AASHTO LRFD 2020 models using the provisions provided in the body as well as in the commentary, ACI 209R-92 model, B3 model, CEB MC90 model, CEB MC90-99 model, and GL 2000 model. The variation of the creep coefficient based on the B3 model and GL 2000 model was significantly higher than that predicted by the other models. As a result, these two models were excluded to show more clearly the variation between the other models. The variation of shrinkage strain and creep coefficient for beam and deck concrete as a function of the selected model highlights once again the challenge of obtaining accurate long-term camber and displacement predictions. It should be noted that ACI Committee 209 has refrained from endorsing a particular model due to a lack of consensus on the set of data that should be used to evaluate different models as well as the statistical indicators that should be used if an agreement on the data set was reached. The majority of the data presented in this report are based on the AASHTO LRFD 2020 provisions provided in the main body. While these provisions together with the selection of the other models for the prediction of modulus at prestress release and 28 days as well as its variation with time, have resulted in reasonably accurate predictions of pre-erection camber as will be demonstrated later, the user is encouraged to explore the use of different models to get a sense for the variation in camber results.

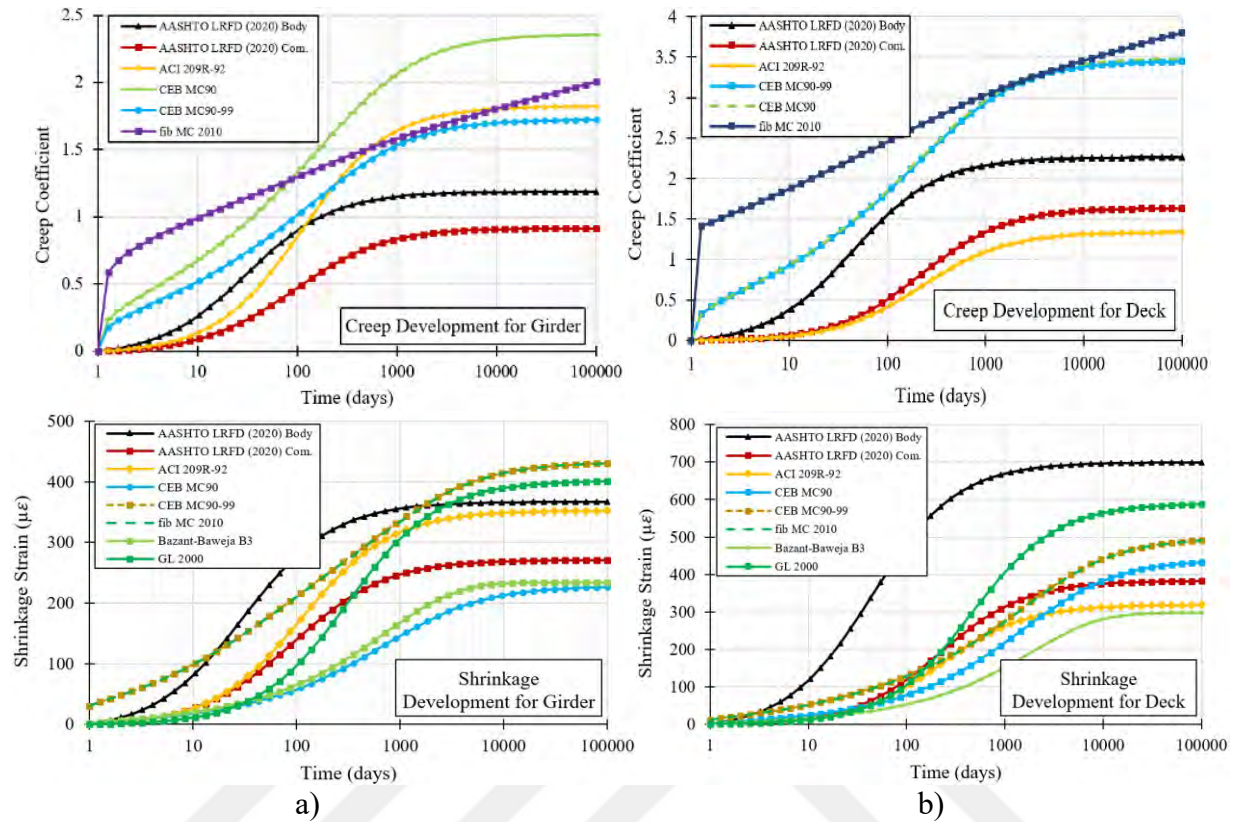


Figure 3.22. Variation in creep coefficient and shrinkage strain based on different shrinkage and creep models for: a) PCBT 54 used in S11 project (Flange width 49 in., $t_c = 1$ day steam curing, $t_{load} = 1$ day, Cement Type I); b) Deck used in S11 project ($t_c = 1$ day steam curing, $t_{load} = 1$ day, Type I cement, mixture proportions are based on a typical deck mix featuring normal weight concrete and fly ash)

A creep test was conducted on a Michigan concrete mix used in one of the projects listed in Table 4.1. The creep test setup is shown in Figure 3.23 and the test was conducted in accordance with ASTM C512. Before loading the cylinders, they were capped using Sulfur to evenly distribute the loads on concrete in accordance with ASTM C617. Figure 3.24 shows the sulfur capped concrete specimens prepared for the creep and shrinkage tests.

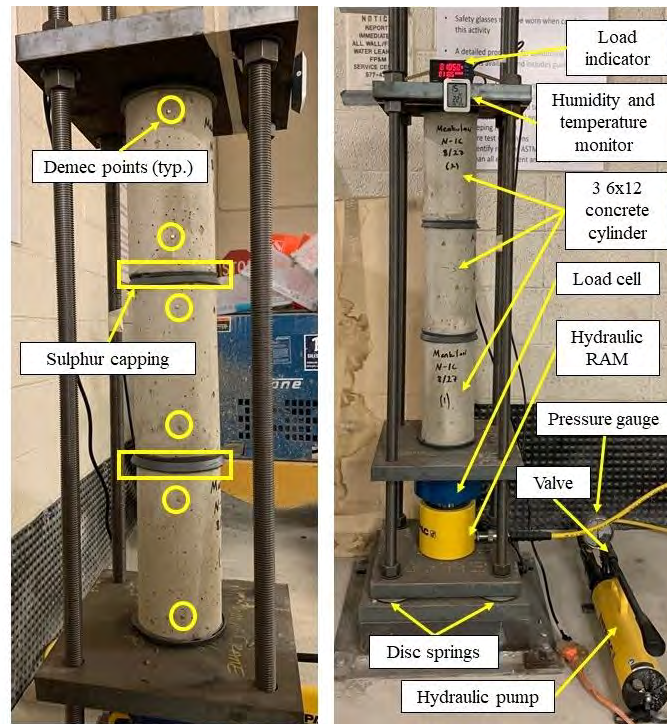


Figure 3.23. Test setup for creep test



Figure 3.24. Sulfur capped concrete samples for creep and shrinkage tests

The cylinders were loaded when they were 15 days old due to logistics related to the sulfur capping of the cylinder ends. Companion cylinders were cast to measure the shrinkage strain so that this shrinkage strain could be deducted from the total strain measured in the creep test so that remaining strain could be attributed solely to creep. The measured data was used to calculate a creep coefficient, which was then compared with predictions based on different models.

Parameters related to environmental such as temperature and relative humidity were measured throughout the experiment. Figure 3.25 shows the measured and average temperature and ambient relative humidity values during the experiment. Based on this, it can be said that while the temperature did not fluctuate much with an average value of 77 °F, there was considerable variation in the ambient relative humidity with average value of 25 %. This value is outside of the range of applicability of some models as mentioned in APPENDIX C, however, for comparison study, it was neglected. Moreover, predictions for the considered models were based on measured compressive strength of concrete at release and 28 days ($f'_{ci} = 8.3$ ksi and $f'_c = 10$ ksi). This information was used to calculate modulus of elasticity at prestress release and 28 days. Such input is required in either all or some prediction models. The initial concrete compressive strength f'_{ci} is also a required parameter in the AASHTO LRFD 2020 creep and shrinkage model. In addition, the mix design for cylinders in question was made available and used when calculating creep coefficients based on a specific model.

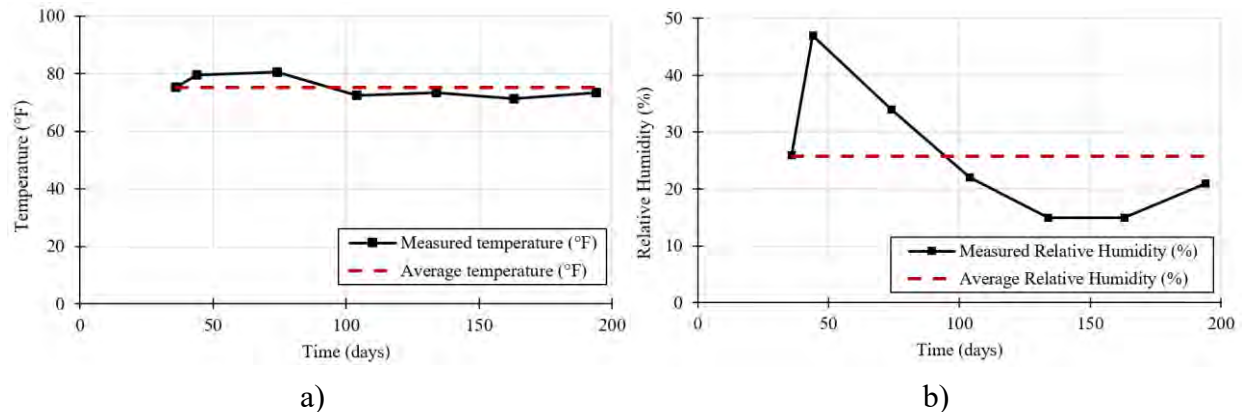


Figure 3.25. Measured temperature and ambient relative humidity in lab environment

The results of this comparison are shown in Figure 3.26. The model based on AASHTO LRFD 2020 main body provisions provided the most accurate prediction for the creep coefficient. However, it should be noted that this conclusion is based solely on the data collected for this one mix.

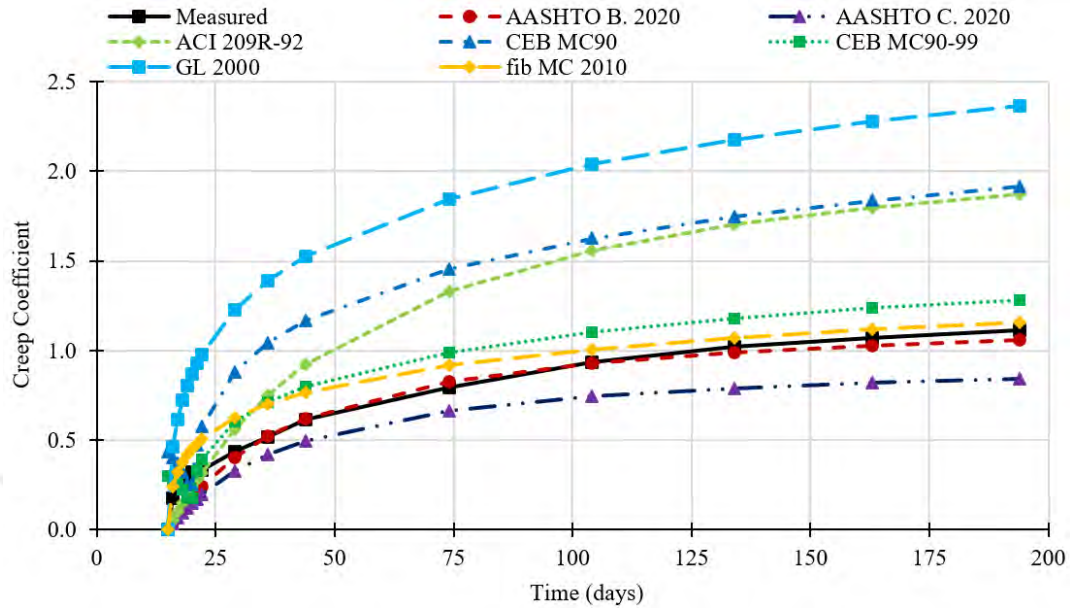


Figure 3.26. Comparison of measured and predicted creep coefficient

Furthermore, the shrinkage data collected from the companion cylinders was compared with predicted shrinkage strains based on different models. The cylinders were steam cured for one day and then allowed to dry for 14 days prior to measuring shrinkage strain since the specimens were loaded in the creep frame after 15 days. The measured shrinkage strain after 15 days was compared with the predicted change in strain after 15 days based on different models. The results are shown in Figure 3.27. The majority of the models provided reasonable predictions with the exception of Bazant-Baweja B3 model and ACI 209R-92 model, which provided lower and upper bounds. However, it can be said that AASHTO LRFD 2020 commentary provision provided the most accurate prediction for the shrinkage strain. The impact of the aforementioned models on long-term prestressed concrete beam camber is investigated in section 8.4 by comparing predicted values with measured ones for the Tiffin Street Overpass and Cooper River (Hinkle 2006) projects for which multiple camber data for several beams were made available.

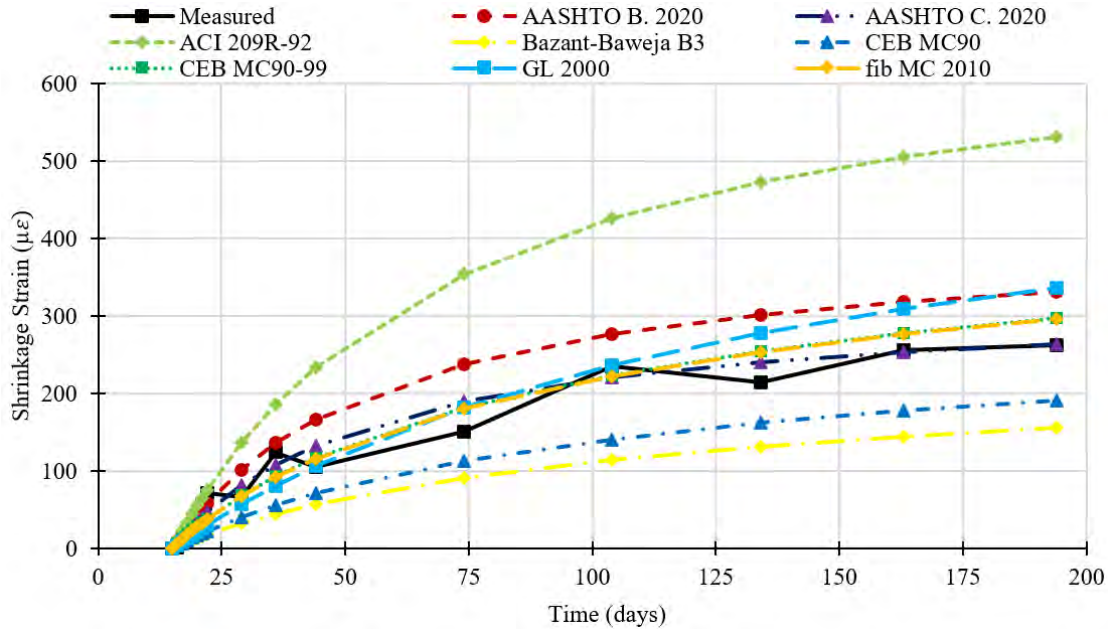


Figure 3.27. Comparison of measured and predicted shrinkage strain

3.8. Strand Relaxation Loss

When strands used in prestressed concrete beams are stressed and kept tensioned for a period of time, the stress on the strands are prone to decrease. This decrease in stress in tendons are called relaxation. Relaxation of strands depend on the stress level, strand grade and type. Nowadays, mostly low-relaxation strands are used in prestressed concrete because the relaxation level of low-relaxation strands are only 25% of stress-relieved strands which have normal relaxation (Naaman 2012).

In this thesis, relaxation loss is calculated based on NCHRP Report 496. AASHTO LRFD 2020 also provides a formula for the calculation of relaxation loss, however it could not be used for the cases where time at transfer is 1 day. Therefore, equation provided in NCHRP Report 496, which is quite similar to one provided in the commentary of AASHTO LRFD 2020 is preferred to be used.

According to AASHTO LRFD 2020, the relaxation loss depends on strand type, strand grade and level of stress on the strands. In the body of the code, the relaxation loss between transfer and deck placement can be estimated using Eqn. 3-118.

$$\Delta f_{pR1} = \frac{f_{pt}}{K_L} \left(\frac{f_{pt}}{f_{py}} - 0.55 \right) \quad 3-118$$

Where:

Δf_{pR1} is the relaxation loss between transfer and deck placement (ksi),

f_{pt} is the stress in prestressing strand immediately after transfer, taken not less than $0.55f_{py}$ (ksi),

f_{py} is the yield strength of strands, typically taken as $0.9f_{pu}$ (ksi),

f_{pu} is the ultimate strength of the strands (ksi),

K_L is the factor accounting for type of steel taken as 30 for low-relaxation stands, and 7.0 for other prestressing strands.

Moreover, a more accurate estimation of relaxation loss at any time can be calculated using Eqn. 3-119 based on NCHRP Report 496.

$$\Delta f_{pR} = \left[\frac{f_{pt}}{K'_L} \frac{\log(24t + 1)}{\log(24t_i + 1)} \left(\frac{f_{pt}}{f_{py}} - 0.55 \right) \right] \left[1 - \frac{3(\Delta f_{pSR} + \Delta f_{pCR})}{f_{pt}} \right] K_i \quad 3-119$$

$$K_i = \frac{1}{1 + \frac{E_p}{E_{ci}} \frac{A_{ps}}{A} \left(1 + \frac{Ae^2}{I} \right) [1 + \varphi(t, t_i)]} \quad 3-120$$

Where:

Δf_{pR} is the relaxation loss at any time (ksi),

K_i is the coefficient that takes into account the transformed section properties,

K'_L is a parameter for strand type, 45 for lox-relaxation steel and 10.5 for other types,

Δf_{pSR} is the loss in the strands caused by shrinkage of concrete (ksi),

Δf_{pCR} is the loss in the strands caused by creep of concrete (ksi).

E_p is the modulus of elasticity of strands (ksi),

E_{ci} is the modulus of elasticity of concrete at release (ksi),

A_{ps} is the total cross-sectional area of prestressing strands (in.²);

A is the cross-sectional area of: a) girder between transfer and deck placement, b) composite structure between deck placement and final time,

e is the eccentricity of prestressing strands with respect to: a) centroid of girder, b) centroid of composite section (in.),

I is the moment of inertia of: a) girder, b) composite structure (in.⁴),

$\varphi(t, t_i)$ is the creep coefficient at time t for the load applied at t_i ,

t_i is the loading time (days),

t is the time of interest (days).

The second term presented in the square parenthesis in Eqn. 3-119 is a reduction due to change in force on the strands due to losses caused by shrinkage and creep. In addition, it should be noted that there may be some relaxation between tensioning and release and ACI Committee 343R-95 suggests including the relaxation loss during the time before the transfer of prestress. However, it was neglected in this thesis.

Alternatively, AASHTO LRFD 2020 allows relaxation loss to be taken equal to 1.2 ksi for low-relaxation strands between prestress transfer and deck placement and, between deck placement and final time. The proposed methodology uses the equation provided in NCHRP Report 496, which is quite similar to the equation provided in the commentary of AASHTO LRFD 2020 for computing strand relaxation loss since it is claimed to provide more accurate estimates. While it is attempted to compute relaxation losses as accurately as possible it should be noted that relaxation losses are rather small compared to losses caused by other effects.

3.9. Elastic Shortening Loss

When the prestressing strands are released, an instantaneous shortening occurs in the beam due to the application of compression force caused by prestressing strands. Depending on cross-sectional properties of the member and magnitude of the prestressing force, the amount of shortening can be estimated. Because of that shortening, initial jacking stress on the strand slightly decreases, which is called elastic shortening loss. The loss due to elastic shortening in pretensioned members can be estimated by the closed formula provided in the body provision of AASHTO LRFD 2020.

$$\Delta f_{pES} = \frac{E_{ps}}{E_{ci}} f_{cgp} \quad 3-121$$

Where:

Δf_{pES} is the elastic shortening loss, ksi,

E_{ps} and E_{ci} are the elastic modules of prestressing strands and concrete at transfer, ksi,

f_{cgp} is the concrete stress at the center of gravity of the prestressing tendons due to prestressing force immediately after transfer at midspan, ksi.

As it can be seen from the Eqn. 3-121 that the process to determine the elastic shortening loss is iterative because the term f_{cgp} which is necessary to calculate elastic shortening loss also depends on elastic shortening loss. AASHTO LRFD 2020 suggests taking an initial estimate of prestressing force as 90% of the jacking force (force on the strands before transfer). With that assumption, analysis is iterated until sufficient accuracy is obtained.

Moreover, AASHTO LRFD 2020 allows using another closed-form formula to avoid the iterative procedure, as indicated below.

$$\Delta f_{pES} = \frac{A_{ps} f_{pj} (I_G + e_m^2 A_G) - e_m M_G A_G}{A_{ps} (I_G + e_m^2 A_G) + \frac{A_G I_G E_{ci}}{E_{ps}}} \quad 3-122$$

Where:

A_{ps} is the strand area, $in.^2$,

A_G is the gross cross-sectional area, $in.^2$,

E_{ps} is the modulus of elasticity of prestressing strands, ksi,

E_{ci} is the modulus of elasticity of concrete at prestress transfer, ksi,

e_m is the midspan strand eccentricity, in.,

f_{pj} is the jacking stress, (stress in the strands before transfer), ksi,

I_G is the moment of inertia of the gross concrete section about the centroidal axis, $in.^4$,

M_G is the moment caused by member self-weight, kip-in,

Since Eqn. 3-122 eliminates the drawback of Eqn. 3-121, it is preferred to be used in this thesis. It should be noted that while calculating the concrete stresses using transformed section properties, the loss due to elastic shortening are implicitly taken into account. This phenomenon

is explained by AASHTO LRFD 2020 section C5.9.3.2.3a by the fact that while utilizing the transformed section properties, strands and concrete are treated together which comprises a composite section, in which both concrete and strands are equally strained in compression by a prestressing force conceived as fictitious external load applied at the centroid of the strands. Thus, in this case, elastic shortening loss should not be deducted from prestressing force. Nonetheless, while determining the effective stress in the strands and calculating the relaxation loss, it should be deducted from the jacking stress.

3.10. Temperature Gradients

For statically determinate systems, uniform or linearly varying temperature over the depth of the cross-section does not lead to change in curvature of the member because all the fibers expand or contract at same amount. On the other hand, if temperature variation is not uniform but varying along the depth of the cross-section, as seen in Figure 3.28, the fibers expand or contract differently with respect to each other. In this case, since each fiber which is attached to adjacent fibers cannot deform freely, curvature changes in the members are observed. This phenomenon is quite similar to differential shrinkage between the concrete and the strands or between the deck concrete and beam concrete.

The temperature gradient used in this report for quantifying its effects on beam deflection was obtained from AASHTO LRFD Bridge Design Specifications (2020) for solar radiation zone III. Temperature gradients used in this thesis have a bi-linear shape for both prestressed concrete and steel beams, as shown in Figure 3.28. Negative temperature gradients can be obtained by multiplying the positive temperature gradient values by -0.30 (AASHTO LRFD 2020) for plain concrete decks. In the temperature gradient analysis, existence of overlay on the deck is ignored. Furthermore, while calculating the effects of temperature gradients on member's deflection, creep

effects were not considered because it was assumed that the temperature gradient would develop over a period of 8h. Thus, creep effects occurring such a short period of time would be negligible (Menkulasi 2018).

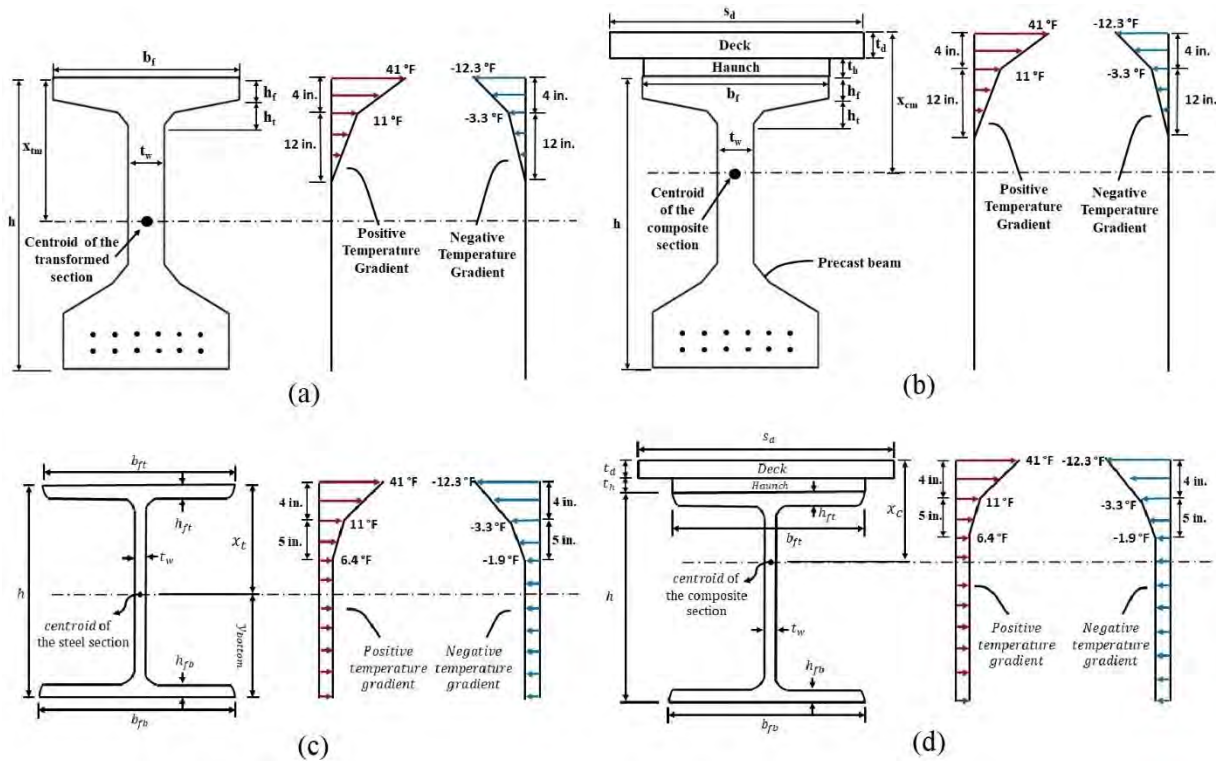


Figure 3.28. Temperature gradients based on AASHTO LRFD 2020 solar radiation zone III for: (a) Non-composite prestressed concrete beam (b) Composite prestressed concrete beam; (c) Steel beam alone; (d) Composite steel beam

Changes in curvature caused by temperature gradients can be calculated by dividing the moments created due to temperature gradients by the flexural stiffness of the section as expressed in Eqn. 3-123. Moments created by temperature gradients are calculated by integrating the stresses over the depth of the section where temperature gradient applies and by multiplying each infinitesimal area of stress block by the corresponding distance to the neutral axis of the section, as shown in Eqn. 3-124.

$$\Delta\phi_{TG} = \frac{M_{TG}}{EI} \quad 3-123$$

$$M_{TG} = - \int_0^h \alpha T(y) E b(y) y dy \quad 3-124$$

Where:

$\Delta\phi_{TG}$ is the change in curvature due to temperature gradient (1/in.),

M_{TG} is the moment created by temperature gradient (kips-in.),

α is the thermal expansion coefficient (1/°F),

T is the temperature gradient, as defined in Figure 3.28 according to AASHTO LRFD 2020 zone III, (°F),

E is the modulus of elasticity (ksi),

b is the width of the infinitesimal area varying along the height, (in.),

y is the distance from centroid of infinitesimal area to neutral axis (in.),

h is the distance from top of the section to the neutral axis (in.).

According to the temperature gradients given in Figure 3.28 , when a beam is subjected to positive temperature gradient, the top flange expands more than other parts of the beam. Hence, it leads to upward deflection. On the other hand, when a beam is subjected to negative temperature gradient, top flange would contract more than the other parts of the beam. Therefore, beam deflects downward. As it can be inferred from Eqn. 3-124, which supports the idea explained above, in both composite or non-composite structures, positive temperature gradient would cause negative moment, thus negative curvatures and negative deflection, which means upward deflection

according to adopted sign convention in this thesis (Figure 3.1). Vice versa can be said for the negative temperature gradient.

The effects of temperature gradients on four prestressed concrete beams which belong to Tiffin Street Overpass Project, are discussed comprehensively in section 4.12 by taking different set of measurements throughout the day. In addition, a study is conducted in section 6.2.6 to quantify the effect of temperature gradients on service camber displacements.

3.11. Limitations of the Proposed Prediction Methodology

The previous sections presented the approach as well as the assumptions made during the development of the proposed prediction methodology. For fully prestressed concrete beams the prediction methodology can be used to predict camber and displacements at any point in time during the life of the beam. For steel beams, the prediction methodology can be used to predict camber and displacements in the composite system. In both cases the non-composite and composite beams are assumed to be prismatic and simply supported. The proposed prediction methodology does not address multi-span continuous bridges or non-prismatic members. It also does not address partially prestressed concrete beams (i.e. beams that are allowed to crack under service loads), although in Michigan such beams are rare due to the aggressive environment created by the cold climate and the treatment of bridge decks with deicing salts. The extension of the prediction methodology to continuous bridges could very well be the scope of future studies. If this is pursued the following would need to be considered:

- 1) Quantification of restraint moments and their impact on beam camber and displacements.

This will depend on the detail adopted by MDOT to create continuity for live loads.

- 2) Unequal spans and different beams for each span. This will require the entry of the relevant information for each span. This is a significant research endeavor as it requires to extend

the capabilities of the computational tool to include time-dependent analysis in statically indeterminate structures.

- 3) Consideration of various construction sequences. For example, in multi-span continuous bridges, the deck is sometimes placed such that the portion over the interior supports is placed later than the rest of the deck to reduce the likelihood of any transverse deck cracking due to negative restraint moments or any continuity diaphragm cracking due to positive restraint moments. This sequence is different compared to the case where the entire deck is placed in one installation in terms of the history of beam camber and displacements.
- 4) Consideration of continuity in pin-hanger connections in terms of how it affects the variation of internal forces if the tool is anticipated to be used in these cases.
- 5) Consideration of any potential cracking in the deck or beams if partially prestressed beams were ever pursued or if it is found that existing prestressed concrete bridges crack under service loads.

3.12. Time-Dependent Multiplier Method

While the algorithm developed based on the proposed methodology (Approach No. 6) is relatively straightforward to use, it does require the input of several variables. When only pre-erection camber is of interest, an alternative method to predict it is to use time-dependent multipliers. The benefit of this method is that it provides a multiplier that is time-dependent and thus removes the limitation imposed by PCI and MDOT fixed value multipliers for any pre-erection time. The disadvantage of this approach is that these multipliers are empirically derived based on the dataset used as part of this research project. In this approach, time-dependent multiplier curves were developed for each bridge project based on results obtained from the proposed methodology. A regression analysis was conducted to capture the effect of some project

specific parameters. The ratio between long-term camber and camber at release was plotted as a function of time. The analysis was conducted up to 90 days. The multipliers are applied to the net camber at prestress release (i.e. the camber due to prestress and beam self-weight). Detailed information about the proposed time-dependent multiplier method is provided in section 8.7.



CHAPTER 4: Beam Camber and Displacement Data Sets

4.1. Introduction

A dataset of measured camber for various prestressed concrete and steel beams used in bridge projects in Michigan was made available by Michigan Department of Transportation, MDOT. The measured camber typically included measured camber at release and measured pre-erection camber. In addition, the dataset included bridge drawings, quality assurance (QA) and quality control (QC) reports. The QA/QC reports included measured concrete compressive strength, measured prestressing force at release, measured unit weight, and measured beam length. The camber and displacement data sets provided by MDOT were evaluated in terms of frequency, time, location, and instrumentation used to measure camber as well as other information provided as part of the data set. This evaluation was done with the purpose that the provided dataset could later be used to assess the accuracy of the proposed long-term camber prediction method.

In terms of frequency, camber and displacement data sets provided by MDOT were supplied at two different times for each project; once after the detensioning of the strands (first set) and another time prior to the erection of the beams at the bridge site (second set). The first set of camber data was measured by the fabricator and was accompanied with information on the measured concrete strength when the strands were detensioned, measured length of beams, measured unit weight, and measured prestressing force. However, not all of this information was provided for each project discussed in this section. MDOT's policy in terms of the time frame for collecting this set of data is that camber be measured at mid-span for each beam no more than 7 calendar days after releasing the prestressing force. The second set of data was measured by agencies hired by MDOT before the beam leaves the precastor's facility for erection (between 14 and 21 calendar days prior to setting the beam seat elevations at the abutments and piers). While

these two sets of data taken at two discrete times were certainly helpful in evaluating the accuracy of the proposed methodology, the research team collected additional data at a higher frequency from a precast plant in Windsor, Canada and Wyoming, Michigan to further validate the proposed prediction methodology and to develop the full camber growth curve for several beams. The precast fabricator in Windsor is Prestressed Systems Inc. (PSI), and in Wyoming is Peninsula Prestress.

In terms of time, ideally, daily measurements should be taken early in the morning to eliminate the effects of radiation-induced temperature gradients and allow collection of consistent data. However, given that the first set of camber data were prepared by the precast fabricator who measured camber sometime after the detensioning of the strands, and that the second set was measured by other agencies employed by MDOT sometime prior to the erection of the beams at the bridge site, it may be difficult to ensure short-term and long-term collection of camber data at a specified period of time during the day. However, as stated earlier, if possible, collection of such data at a consistent time during the day will help eliminate the effects of radiation-induced temperature gradients and allow collection of consistent data. To quantify the effects of radiation-induced temperature gradients, the research team collected data on several beams at the PSI Windsor plant and Peninsula Prestress plant several times during a day including early in the morning, mid-morning, and noon. A detailed discussion about the effect of solar radiation on beam camber displacement is provided in sections 4.12 and 6.2.6.

In terms of location, the first set of camber data prepared by the fabricator were measured at mid-span of the beam. It was assumed that the beam was supported as provided in the drawings with supports located a distance L_t away from the ends of the beam. In cases when the support conditions were not shown in the drawings it was assumed that the beams were supported as

indicated in MDOT's Standard Specifications for Prestressed Concrete Beams, which states: "Support stockpiled beams across the full width on two battens, each greater than 4 inches wide. Do not support beams at more than two points. Use battens to hold beams off the ground over the full length. Place battens in from the beam ends no greater than $1\frac{1}{2}$ times the depth of the beams, or 3 feet, whichever is less."

The second set of data were collected at multiple points along the span of the beam as outlined in the document titled "Special Provision for Structure Survey during Construction" prepared by MDOT. This document requires the reporting of the location of the supports during storage as this information affects the magnitude of displacements due to self-weight and consequently the magnitude of camber. However, such information was not provided. As a result, similar to camber at prestress release predictions, long term-camber predictions used the support distance L_t discussed above.

In terms of instrumentation, information regarding instruments used by the fabricator to measure short-term camber and those used by the other agencies to measure long term camber was not provided.

The information provided by the precast fabricator contained information on actual beam length, magnitude of prestressing force, concrete unit weight, and measured concrete compressive strength at release. This information was used to quantify the variability between specified and measured properties because such variation has been reported to be one of the main reasons for the disparity between predicted and measured camber.

In addition to the data provided by MDOT and collected by research team, some of the data available in the literature was used such as Branson et al. (1970), Tadros et al. (1977), Bradford et al. (1991), Hinkle (2006) and Fan et al. (2010). While data provided by Hinkle (2006)

includes only periodic camber measurements up to pre-erection, other data includes service camber and displacement measurements.

The provided short-term (first set) and long-term (second set) camber and displacement data together with the ones that was collected by research team that was available in the literature, were used to evaluate the accuracy of the proposed methodology. The following sections provide detailed information on the camber and displacement data sets provided by MDOT, camber data collected by the research team at the PSI Windsor plant and Peninsula Prestress plant, and camber data that was available in the literature and the evaluation of the proposed prediction methodology.

4.2. Camber and Displacement Data Sets Provided by MDOT

Camber and displacement data sets were provided by MDOT on 16 projects, which are shown in Table 4.1. 15 projects out of 16 are composed of prestressed concrete beams and one project is composed of steel beams. There were additional projects provided by MDOT, however, they were not included in this table because provided information was not sufficient to conduct analysis. For example, there are some projects in which the camber and displacement data sets are provided, but the girder drawings are missing, and vice-versa. Out of 15 prestressed concrete beam projects, only 7 of them includes long-term measurements up to pre-erection. Although service camber measurements, measurements after deck is cast, are provided for couple of projects, they could not be used due to insufficient information to be able to conduct the analysis. Table 4.1 is a short description for each project and the data that was made available.

Projects M5, Concord Avenue, I75-S09, I75-S10, I75-S12, French Road over I-94, Springwells and Livernois consisted of two spans and project Shepherd Road consisted of three spans, and each span was considered as a separate project for the purpose of the analysis presented herein if girders showed some differences. For example, the results for project I75-S10 are

presented as I75-S10-1 and I75-S10-2. The same applies for project I75-S12. However, since the girders at each span in M5 project shows the same features, they were not considered as a separate project.



Table 4.1. Camber and displacement data sets

Project name	Girder type and depth (in.)	No. of beams/data ⁹	Span length (ft.)	Deck thickness (in.)	Center-to-center spacing (ft.)	Strand configuration ¹	Available camber data ²	Fabricator ⁷	f'_{ci} (psi) ⁴	f'_c (psi) ⁴	f'_{deck} (psi) ⁴
M5 1-2	Bulb tee (42)	6/6	144.33	6	6.00	H@0.4L	R & P	PP	6100	8000	4000
M57	MI 1800 (71)	8/8	137.80	9	7.92	H@0.4L	R	PP	6000	6000	4000
M89	Spread Box (17)	6/6	28.25	9	8.00	D	R & P	PP	4300	5000	4000
M311	Bulb tee (60)	6/6	147.33	9	7.46	D	R & P	PSI - LLC	7000	8500	4000
TSO ⁸	Bulb tee (63)	4/4	98.33	8.85	8.20	H@0.40-0.33L	R & P	PSI	6100	7200	NA ¹⁰
Conc. Ave. 1	Bulb tee (48)	8/4	115.50	9	6.50	D	R	PP	7200	9000	4000
Conc. Ave. 2	Bulb tee (48)	8/8	91.50	9	6.50	D	R	PP	5200	9000	4000
I75-B05	AASHTO Type II (36)	9/9	39.50	9	5.83	-	R	PSI - LLC	6000	8000	4000
I75-S05	Bulb tee (60)	11/11	152.50	9	9.04	D & H@0.4L	R & P	PSI - LLC	7000	10000	4000
I75-S09-1	Bulb tee (48)	9/9	115.54	9	7.00	D	R	PSI - LLC	7000	10000	4000
I75-S09-2	Bulb tee (48)	9/9	112.79	9	7.00	D	R	PSI - LLC	7000	10000	4000
I75-S10-1	Bulb tee (42)	8/8	80.83	9	8.88	H@0.4L	R & P	PSI - LLC	7000	10000	4000
I75-S10-2	Bulb tee (42)	8/8	97.33	9	8.88	D & H@0.4L	R & P	PSI - LLC	7000	10000	4000
I75-S11	Bulb tee (54)	9/9	136.00	9	8.42	D & H@0.33L	R & P	PSI - LLC	7000	10000	4000
I75-S12-1	Bulb tee (42)	7/7	96.75	9	8.08	H@0.4L	R & P	PSI - LLC	7000	10000	4000
I75-S12-2	Bulb tee (42)	7/7	106.17	9	8.08	H@0.4L	R & P	PSI - LLC	7000	10000	4000
French Rd. over I-94 1-2	Bulb tee (42)	18/14	99.04	9	7.00	H@0.4L	R & P ⁶	PP	6000	8000	4000
Shepherd Rd. 1-3	Spread Box (21)	10/4	55.92	NA ¹⁰	7.00	D	P ³	PP	6500	7000	NA ¹⁰
Shepherd Rd. 2	Spread Box (21)	5/5	62.17	NA ¹⁰	7.00	D	P ⁵	PP	6500	7000	NA ¹⁰
I75-Springwells-1	Bulb tee (48)	11/2	86.67	9	9.15	D & H@0.4L	R	PP	7000	8000	4000
I75-Springwells-2	Bulb tee (48)	11/2	74.67	9	9.15	D & H@0.4L	R	PP	7000	8000	4000
I75-Livernois-1	Bulb tee (48)	12/0	111.00	9	6.63	D & H@0.4L	NA ¹⁰	PP	7000	8000	4000
I75-Livernois-2	Bulb tee (48)	12/2	96.75	9	6.63	D & H@0.4L	R	PP	7000	8000	4000
Saginaw County	Bulb tee (42)	12/12	118.9	NA ¹⁰	6.17	D & H@0.4L	R & P	PP	7000	10000	NA ¹⁰
M20	I-shaped steel beam (70)	20/20	172-190 ¹¹	9	10.0-10.5 ¹²	Not applicable	PRC & PC ¹³	NA ¹⁰	Not applicable	Not applicable	4000
Cooper Riv. Br. ¹⁴	Bulb tee (79)	14/14	128-136	NA ¹⁰	NA ¹⁰	D	Periodic	NA ¹⁰	6100	9000	NA ¹⁰

1. H: Harped, D: Debonded.
2. R: Camber at release, P: Camber at pre-erection.
3. Camber measurements were recorded at the precast facility 9 days after beams were cast.
4. Specified concrete strength.
5. Camber measurements were recorded at the precast facility 1-to-3 days after beams were cast.
6. Provided measured pre-erection camber data could not be used due to insufficient information.
7. PP: Peninsula Prestress, PSI: Prestress Services Industries.
8. TSO: Tiffin Street Overpass, Canadian Project.
9. Represents number of beams for which camber and displacement data was available.
10. Not available.
11. There are 4 spans whose lengths vary.
12. There are 5 beams in transverse direction whose spacings vary.
13. PRC: pre-construction camber (without beam self-weight), PC: post-construction camber.
14. Data was obtained from Hinkle (2006).

The prestressed concrete beams for MDOT bridge projects were fabricated by either Peninsula Prestress or PSI Decatur. Based on information supplied by MDOT, the following procedures/policies are used during the fabrication of the prestressed concrete beams by various precast fabricators who supply prestressed concrete beams for bridge projects in Michigan.

1. Casting of Concrete: Concrete is prepared and mixed in the precast facility and transported to the forms. Several fresh concrete tests are conducted to ensure compliance with the specifications such as: concrete temperature, slump, and air content. Also, concrete cylinders are prepared for compressive strength testing at release and 28 days. If the results from fresh air concrete tests deviate from the specifications, then an attempt can be made to adjust concrete such that it complies with specification requirements within the time permitted in the MDOT Standard Specifications to avoid rejected concrete. For example, concrete can be adjusted in truck mixers by doing an additional 30 revolutions at mixing speed. During concrete placement, the steam is shut off and is turned back on after initial set of the concrete to help with accelerated curing. At the end of the concrete pour, tarps are secured over the beams for moisture retention and insulating purposes. Whenever any type of external heat source is used, thermocouples are used to record the concrete temperature (maximum concrete temp allowed is 150 °F).
2. Curing: The curing techniques used by various precast fabricators that supply prestressed concrete beams for bridge projects in Michigan are summarized in Table 4.2. The curing process is completed once the specified f'_{ci} is reached. The MDOT specifications for beam curing are provided in section 708.3 of Standard Specifications for Construction.

3. Detensioning the strands: Prestressing force is transferred only after the specified concrete compressive strength at release is reached. Detensioning of the strands is done gradually according to a specified sequence to minimize eccentricities.
4. Beam Storage: Once the strands are detensioned, the beam is stored in the yard until the time comes to ship it.

Table 4.2. Curing techniques used by precast beam fabricators that supply prestressed concrete bridge beams for MDOT

Precast Fabricator	Curing Technique
Peninsula Prestress	Live steam. The steam lines run on the outside bottom of the beam beds
Mack, Zilwaukee	Boiler heating system (oil) run along outside bottom of beam beds*
Mack, Kalamazoo	Radiant heat or live steam is used along outside bottom of beam beds
PSI-Decatur	<i>Sure Cure System</i> (electrical curing system) programmable to ensure the concrete follows a consistent time/temp curve
Spancrete	Live steam
Kerkstra Precast	Indoor prestressing beds. Curing technique uncertain*
*There are currently no active projects at this plant.	

4.3. Camber and Displacement Data Sets Collected from Precast Fabricators

In addition to the data provided by MDOT, the research team collected additional short-term and long-term camber data from the PSI Windsor plant located in Windsor, Canada and Peninsula Prestress plant located in Grand Rapids, Michigan. The measurements for the former plant include four prestressed concrete beams used in the Tiffin Street Overpass (TSO) project. This bridge has a span of 96 ft. and 9 in., and features a total of seven prestressed concrete beams. However, due to the tight casting and shipping schedule, camber data was collected only for four beams. PSI supplied partial bridge drawings and specifications so that the research team could collect the necessary data to predict short-term and long-term camber. Camber data was collected twice a week for a period of four weeks, which was approximately the period of time that the

girders stayed in the precast facility prior to being shipped to the bridge site. The number of camber data points collected per beam varied from four to eight depending on when the beam was cast. This increase in the number of camber data points compared to the two data points per beam supplied for the MDOT projects provided an opportunity to create a camber growth curve for the beams in question and to validate the long-term camber predictions at additional times. In addition, for all four beams several measurements were taken two days before the girders were shipped to the construction site at various times during the day such as morning, mid-morning, and noon to quantify the effects of radiation induced temperature gradient.

Moreover, the measurements taken from the Peninsula Prestress plant in Michigan includes ten box beams used in the Shepherd Road over Chippewa River and one beam for the Saginaw County over Lake State Railway Company. In the first one, Shepherd Road project, the effect of solar radiation on beam camber and displacements was investigated. Therefore, all measurements were taken on the same day at various times during the day such as early in the morning (8.00 a.m.), in the mid-morning (11.00 a.m.) and in the afternoon (2.00 p.m.). The project consists of three spans in which the first and the third spans have identical span length with 57 ft. and 9 in. On the other hand, the middle span features 62 ft. and 6 in. span length. Out of these ten box beams, five of them belong to either span 1 or span 3, and remaining five belongs to span 2. All ten beams have same cross-sectional properties with different prestressing layout. Although the project consists of 15 box beams, measurements for only ten beams were able to be collected due to storage conditions of beams. The second one, Saginaw project, composed of single span with length 118 ft. and 11 in. For this project, (12) 42" top flange width 49" height prestressed concrete bulb tee beams were produced. Due to tight storage condition of the beams, measurements for only one

beam could be collected. Measurements were collected periodically to explore the camber growth with time.

The detailed information about how the girder was produced and how the measurements were taken is provided for Tiffin Street Overpass project. As stated earlier, the Tiffin Street Overpass project features a total of seven 98 ft. long CPCI 1600 prestressed concrete beams whose properties and dimensions are illustrated in Figure 4.1. As it can be seen from the Table 4.1, the prestressed concrete beams did not feature any debonded strand, but did have three sets of harped strands.

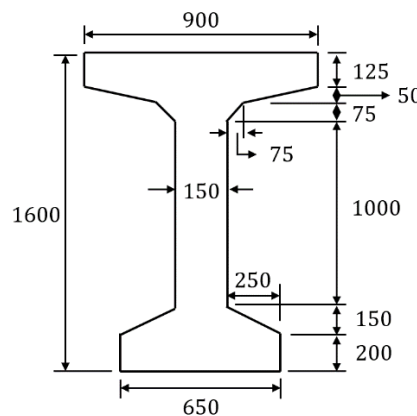


Figure 4.1. Nominal dimensions (in mm) for CPCI-1600 prestressed concrete beams

The following procedure/policy was used during fabrication of girders:

1. Casting of Concrete: Concrete was prepared and mixed in the precast facility and transported to the forms. Several fresh concrete tests were conducted to ensure compliance with the specifications such as: concrete temperature, slump, and air content. Also, concrete cylinders were prepared for compressive strength testing at release and 28 days. If the results from fresh air concrete tests deviate from the specifications, concrete is rejected.

2. Curing: A minimum of four days moist curing is typically used. During the moist curing and moisture retention period, the members may be exposed to ambient condition for no more than a total of 3 hours. The concrete temperature during production, moist curing and moisture retention period shall not fall below 50 °F before the concrete has reached 75% of the specified 28 day compressive strength. Once the concrete reaches 75% of specified 28 day compressive strength, then temperature control does not apply.
3. Detensioning the strands: Prestressing force is transferred only after the specified concrete compressive strength at release is reached. Detensioning of the strands is done gradually according to a specified sequence to minimize eccentricities.
4. Additional Curing: After the initial four days moist curing and moisture retention period the girder can be moved outside without any additional curing if the ambient temperature is 5 °C or higher. If the member is moved outside, and the outside ambient temperature is below 5 °C the girder is protected via temperature control using an outside curing chamber for a minimum of 3 additional days. In this case, the total curing period becomes 7 days.
5. Beam Storage: Once the curing process is completed, the beam is stored in the yard until the time comes to ship it. Deep I-beams are braced for stability.

The difference between the fabrication practices used in the PSI Windsor plant and those used by precast fabricators in Michigan is primarily in the initial curing and post curing process. The predominant curing technique in Michigan appears to be steam curing and the curing process is complete once the strands are detensioned. Whereas in the PSI Windsor plant, an initial minimum four days moist curing is used, which may be accompanied by an additional three day curing depending on the outside ambient temperature, when the beam is moved from the prestressing bed. While only the Michigan data will be used to provide regional calibrations to

improve camber predictions, the PSI Windsor data was used to further validate the robustness of the proposed prediction methodology by obtaining additional data points per beam. Additionally, the PSI Windsor data was used to quantify the influence of solar radiation induced temperature gradients on camber.

The first camber measurement for each girder was taken a few days after the girder was removed from the moist-cure chamber and placed into storage. Camber was measured using surveying equipment, which featured a differential level, tripod, a custom made measuring rod, and a bearing block for the rod. The beams were placed on wooden blocks positioned approximately 3 feet from each end. The custom made measuring rod replaced the standard surveying rod, which was too tall to fit between the top and bottom flanges of the beam and featured divisions of up to 1/8 in. as opposed to the custom made rod, which provided divisions up to 1/16 in. The custom made rod was made using a threaded steel rod, which was fitted with a scale and bullseye level. The scale's precision was 1/16 in. The bearing block for the rod was made using dimensional lumber, which was saw cut to fit the bottom flange of the CPCI-1600 Girders being measured and is illustrated in Figure 4.2b. This provided a solid point to place the measuring rod and obtain consistent measurement across the length of the girder. Figure 4.2 shows the basic layout of the girder and differential level used to take measurements. The sight was first leveled using the leveling bubble on the differential level. After leveling the sight, a benchmark reading was taken at position 1. Beam elevations were then recorded at the ends, quarter-points, and at midspan (positions 2 through 6 in Figure 4.2). Then a final benchmark reading was taken at position 7 (same as position 1) to ensure the level did not move during measurements. This procedure was repeated for each beam. Beam elevations were then used to calculate camber at quarter points and mid-span. Since for this project camber measured by the fabricator at release

was not provided, the first set of measurements, which was taken typically 2 days after the release of the prestressing force, was used to indicate camber at release. Additional information about the solar radiation study can be found in section 4.12. In addition, section 8.4 provides a comparison study between measured and predicted values for Tiffin Street Overpass project.

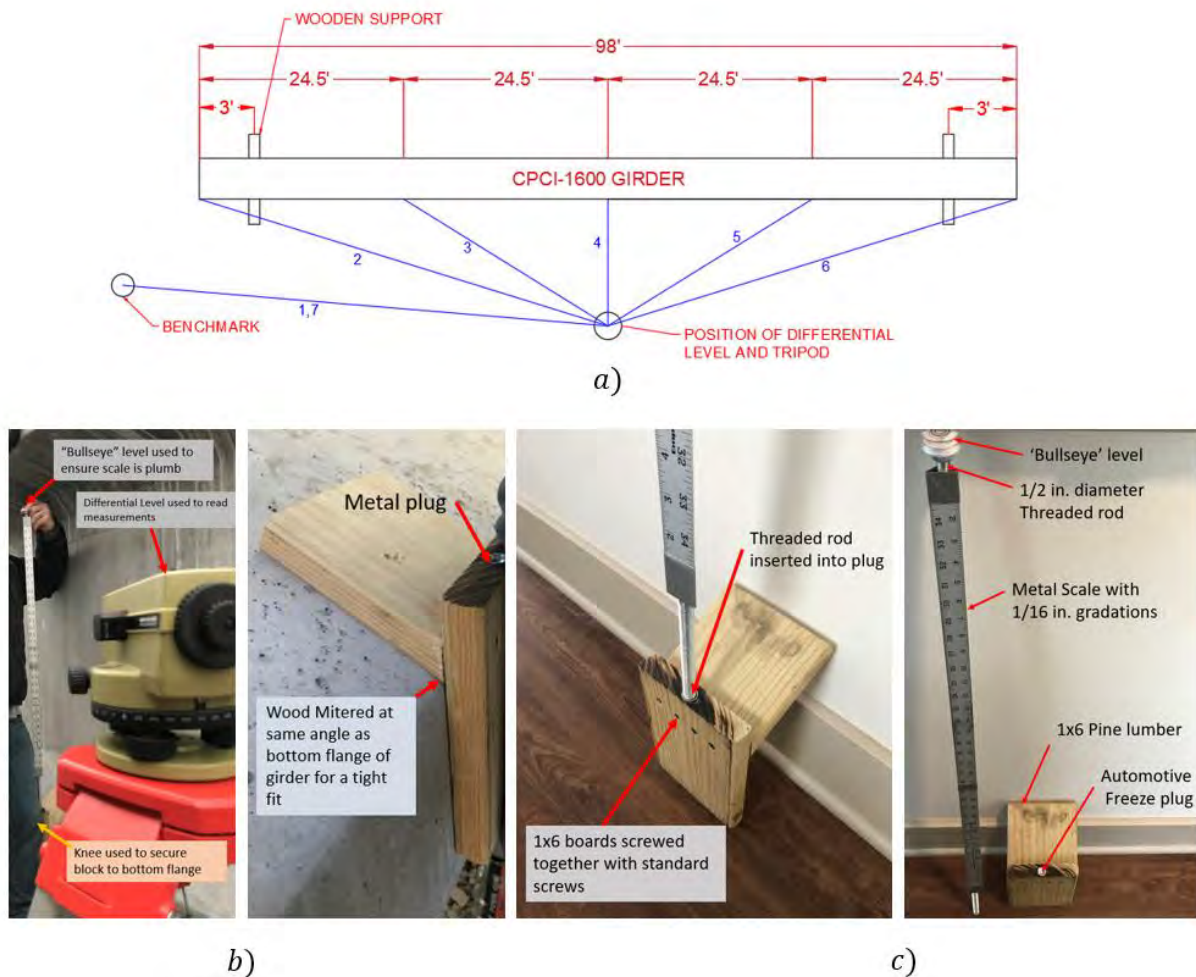


Figure 4.2. a) Illustration of methodology for measuring camber in four CPCI-1600 prestressed concrete beams, b) photograph showing measurement setup and device made for taking consistent measurements across beam bottom flange, c) photograph showing device used to match the bottom flange of the beam and to receive the measuring rod.

Similar to Tiffin Street Overpass project, a set of measurements were taken for Shepherd Road projects' beams. Since this project utilizes box beams, the wood piece shown in Figure 4.2c cannot be used. Instead, the research team considered taking measurements relative to bottom of

the box beams by using a scissor jack and a flat wood piece, as illustrated in Figure 4.3b. Since the bottom of the beams were perfectly smooth, consistent measurements were able to be obtained. The same method in Tiffin Street Overpass project was employed while measurements were taken for this project as well. Furthermore, due to tight storage condition of beams and obstacles in the site, measurements for only 5 beams could be collected.

The main aim of this trip was to observe the temperature gradients on girder camber rather than exploring the camber growth with respect to time. Therefore, on the day when site trip was done, three different sets of measurements were collected at different times of the day: early in the morning, around 8.00 a.m., in the mid-morning, around 11.00 a.m., and in the afternoon, around 1.00 p.m. Unfortunately, the weather on day the trip was done was not a sunny day, hence girders could not be exposed to direct solar radiation. Therefore, the effect of temperature gradients on girders could not be observed very well. Additional information about the solar radiation study can be found in section 4.12.

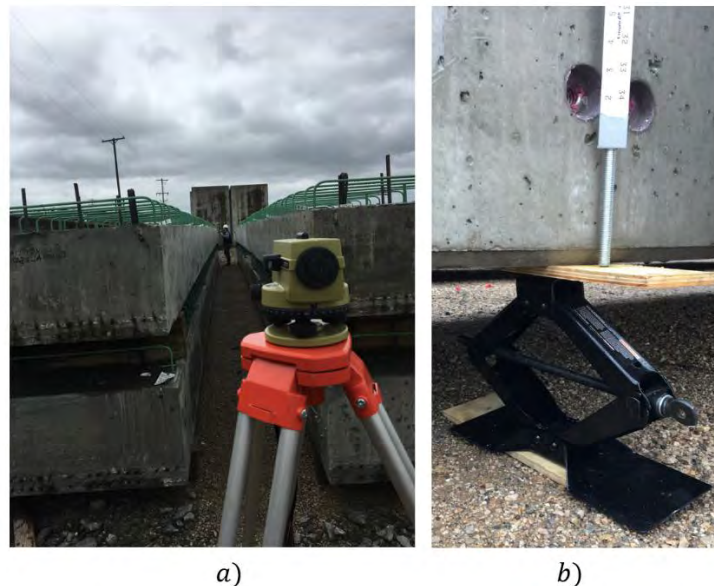


Figure 4.3. a) Photograph showing the storage condition of box beams, b) Photograph showing the bottom of the girder where the measurements are taken relative to

Finally, another periodic camber measurements for the Saginaw Bridge project were collected by research team. There were (12) 42" x 49" prestressed concrete bulb tee beams in this project whose lengths are 118 ft. and 11 in. (Figure 4.4). Due to tight storage conditions, only one set of periodic measurements could be collected for one beam with one week interval. The beams were cast between January 15, 2021 and January 27, 2021. The one that was used to obtain camber measurements was cast on January 22, 2021 and measurements were collected until the end of March 2021. Additional information about this project can be obtained from Table 4.1.



Figure 4.4. 42" x 49" Bulb Tee Beams used in Saginaw Bridge Project

4.4. Camber and Displacement Data Sets Obtained from the Literature

In addition to the database provided by MDOT and collected by research team, additional data available in the literature was utilized to further validate the proposed prediction methodology (Approach No. 6). The presented data up to this section, i.e. ones provided by MDOT and collected by research team, includes either camber at release or pre-erection camber. There was no useful data for the service camber that was made available by MDOT. Therefore, to be able to validate the proposed prediction methodology for post-erection stage i.e. after deck is cast, some of the researcher's data, Branson et al. (1970), Tadros et al. (1977), Bradford et al. (1991), and Fan et al.

(2010), was utilized. In addition, to strengthen the validation of proposed prediction methodology, additional data provided by Hinkle (2006) was also used although it contains camber measurements only up to pre-erection. A short summary about the project information and camber and displacement measurements for the data obtained from literature is provided below and summarized in Table 4.3.

The data provided by Branson et al. (1970) includes 5 prestressed concrete beams which were made up of lightweight aggregate concrete where the unit weights were about 120 pcf. However, the slabs were composed of normal-weight concrete whose unit weights were about 145 pcf. Full camber and displacement histories for these 5 beams, that is from prestress release to 560 days, were provided. The beams were steam-cured until prestressing release which were about 2 – 3 days. In these beams, 30- $\emptyset$ 0.5 in. low-relaxation strands in which 8 of them were harped at 0.4L where L is the beam length were used. The beam height and length, and deck thickness were given as 45 in., 86 ft. and 7 in., respectively. Center-to-center beam spacing was designated as 7 ft. for inner girders. For the fascia girders, average tributary width was given as 6 ft. and 6 in. Branson et al. (1970) stated that the mean value for ambient relative humidity was specified as 70% for the Iowa State with upper and lower bound of 66% and 79%. Since no information about the curing type and duration was provided for the decks, it was assumed that they were 1 day moist-cured in the validation study. While the measured concrete compressive strengths for these beams varied between 4.4 ksi and 5.2 ksi, the design strength for the decks was specified as 3.5 ksi. The decks were cast about 9 weeks after prestress transfer. For the prestressing steel used in these beams, it was stated that the ultimate strength, yield strength and jacking stress were 270 ksi, 250 ksi, and 190 ksi, respectively. Additional information can be obtained from Table 4.3 and Branson et al. (1970).

The data provided by Tadros et al. (1977) includes one prestress concrete beam where full camber and displacement data from prestress release up to 150 days was provided. The beam used in this study was a type of rectangular section with 6 in. width and 10 in. height and 12 ft. span length. After the beam was cast, it was cured for 7 days, and after curing period terminated, prestressing strands were released. The ultimate strength and jacking stress were provided as 280 ksi and 215 ksi, respectively. The prestressing strand area was provided as 0.306 in.² and modulus of elasticity of strands was given as 27400 ksi. 41 days after the beam was cast, the deck was cast on propped structure. In other words, the deck self-weight was intended to be carried by composite structure (shored construction). Although the proposed prediction methodology assumes unshored construction, for this specific case, the algorithm was adjusted such that the deck self-weight was carried by composite structure rather than beam itself. Similar to beam, the deck was also cured for 7 days and props were removed. 53 days after the beam was cast, superimposed concentrated loads of 5.83 kips each were applied and sustained at the third point. The deck width and thickness were designated as 2 ft. and 2.5 in., respectively. Although no information was provided for the girder and deck concrete compressive strengths, moduli of elasticity values were provided for them as 4610 ksi and 3640 ksi, respectively. Since no information about the ambient relative humidity of the environment where the experiment was conducted was provided, it was assumed 70% in the analysis. Additional information can be obtained from Table 4.3 and Tadros et al. (1977).

The data provided by Bradford et al. (1991) includes 4 composite steel beams with different loading conditions and stud intervals. While two of the beams were loaded with sustained uniformly distributed load of 515 plf in addition to self-weight of composite structure, other two were tested under self-weight effect only. The stud interval of the beams to ensure horizontal shear transfer between steel beam and concrete deck varied between 7.87 in. and 23.62 in. The type of

the steel beams used in this experiment was 7.87 in. UB25-4. The span length, deck width and thickness were given as 19.4 ft., 39.4 in. and 2.76 in., respectively. It was stated by Bradford and Gilbert (1991) that there was a relative movement between bottom the of the deck and top flange of the steel girder, however it was reported that it was not significant. Although the proposed prediction methodology assumes that there is a perfect bond between deck and girder, it did not lead to significant errors in the validation study due to limited amount of slippage between deck and girder as reported by Bradford and Gilbert (1991). In the experiment, the average measured concrete compressive strength for decks was specified as 4.51 ksi, and it was obtained 10 days after the decks were cast. Since no information was provided about curing period of the decks and the relative humidity of the environment where the experiment was conducted, it was assumed to be 1 day and 75%, respectively, when proposed prediction methodology was used. Additional information can be obtained from Table 4.3 and Bradford and Gilbert (1991).

The data provided by Hinkle (2006) includes camber measurements up to pre-erection. Periodic measurements for 14 79" deep bulb tee beams were provided. Two sets of beam sample were provided: Beam Sample 1 and Beam Sample 2. The beam lengths were 128 ft. and 136 ft. for the first and second sets of beams, respectively. Specified concrete compressive strength at release and at 28 days for the girders were given as 6.1 ksi and 9 ksi, respectively. Measured concrete compressive strengths were also reported by Hinkle (2006). Additional information can be obtained from Table 4.3 and Hinkle (2006).

The data provided by Fan et al. (2010) includes two steel beams with cast-in-place decks. It was stated before the deck was cast, the girder was propped. Therefore, the deck self-weight was carried by composite structure, not the girder itself. Although the proposed prediction methodology assumes unshored construction, that is the girder is unpropped when deck was cast,

for this specific study, the algorithm was adjusted such a way that the deck self-weight was carried by composite structure. The decks were cured 7 days. After the termination of curing period, the props were removed and a uniformly distributed superimposed load, 427 plf, was applied to the composite structure and kept for 3 years. The only difference between the tested two beams was the concrete compressive strengths of the decks. Measured 28 days concrete compressive strengths of the decks were reported as 3.55 ksi and 4.93 ksi. During the experiment, it was stated that the relative humidity of the environment where the beams were tested varied between 45% and 90% with an average value of 68%. The span length of the beam, and the thickness and width of the deck were provided as 13.12 ft., and 2.36 in. and 23.62 in., respectively. Additional information can be obtained from Table 4.3 and Fan et al. (2010).

Table 4.3. Summary of the database obtained from literature

Source	Beam type	# of beams	L_G (ft.)	t_d (in.)	s_d (ft.)	f'_{ci} (ksi)	f'_d (ksi)
Branson et al. (1970)	Prestressed C.	5	86	7	6.5-7	4.4-5.2	3.5
Tadros et al. (1977)	Prestressed C.	1	12	2.5	2	NP [†]	NP [†]
Bradford et al. (1991)	Steel	4	19.4	2.76	3.28	NA [*]	4.51
Hinkle (2006)	Prestressed C.	14	128-136	NP [†]	NP [†]	6.1	NP [†]
Fan et al. (2010)	Steel	2	13.12	2.36	1.97	NA [*]	3.55-4.93

L_G = beam length; t_d = deck thickness; s_d = girder tributary width; f'_{ci} = girder concrete compressive strength at release; f'_d = deck concrete compressive strength.
[†]Not provided; ^{*}Not applicable

4.5. Relationship between Specified and Measured Concrete Compressive Strength

Since the modulus of elasticity is typically calculated using empirical formulas that are a function of the compressive strength of concrete, it could be deduced that an accurate estimation of the actual concrete compressive strength helps improve the accuracy of the estimation for the modulus of elasticity and consequently camber predictions. However, this is based on the assumption that the empirical equation used to calculate modulus of elasticity as a function of compressive strength is accurate. The equation used in AASHTO LFRD Bridge Design

Specifications (2020) for the modulus of elasticity has been obtained based on curve fitting techniques. The scatter in data is large, however, in general this equation has been considered to be adequate for design purposes.

Since measured compressive strength data was made available as part of the quality control and quality assurance reports, the relationship between the specified concrete compressive strength at release, $f'_{ci\text{specified}}$, and the measured concrete compressive strength at release, $f'_{ci\text{measured}}$, was investigated (Figure 4.5a). Additionally, measured concrete compressive strength data when beams were shipped from the precast facility to the bridge site were also available for some projects. When the beams were shipped to the bridge site, the age of beams was between 28-30 days old. Therefore, measured compressive strength data when the beams were shipped were used to establish a relationship between measured and specified compressive strength at 28 days (Figure 4.5b). The average ratio between measured and specified compressive strength at prestress release was calculated for beams fabricated by three companies: Prestress Services Industries LLC, Peninsula Prestress, and PSI Windsor (Canadian Company). Since only Prestress Services Industries LLC and Peninsula Prestress fabricate beams for bridge projects constructed in Michigan, only data from these companies were used to calculate an average overstrength factor for the concrete compressive strength at release. This overstrength factor (i.e. average ratio of $f'_{ci\text{measured}}/f'_{ci\text{specified}}$) was 1.21 and the coefficient of variation (COV) was 14%. Similarly, the average ratio between measured and specified compressive strength at 28 days ($f'_{c\text{measured}}/f'_{c\text{specified}}$) for the beam fabricated by Prestress Services Industries LLC was 1.21 and the COV was 11%. These overstrength factors are included as an option in the algorithm (Approach No. 6) so that the user can adjust specified compressive strengths accordingly for improved camber predictions.

$$f'_{ci_{measured}} = 1.2f'_{ci_{specified}} \quad 4-1$$

$$f'_{c_{measured}} = 1.2f'_{c_{specified}} \quad 4-2$$

Where:

$f'_{ci_{measured}}$ and $f'_{ci_{specified}}$ are the measured and specified initial concrete compressive strength at release, respectively,

$f'_{c_{measured}}$ and $f'_{c_{specified}}$ are the measured and specified initial concrete compressive strength at 28 days, respectively.

It should be noted that the expressions in Eqns. 4-1 and 4-2 are specifically for Michigan concrete, and may not be necessarily accurate all the time because the available data for measured initial and 28 days concrete compressive is very shallow.

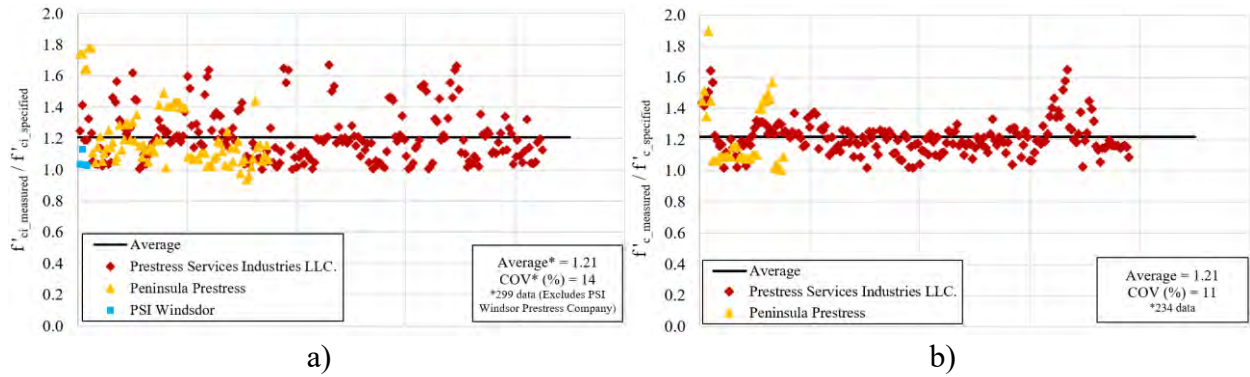


Figure 4.5. Ratio of measured over specified: a) initial concrete compressive strength, b) 28 day concrete compressive strength

Furthermore, Rizkalla et al. (2013) and Rosa et al. (2007) also proposed a relationship between measured and specified concrete compressive strength at initial and at 28 day shown in Table 4.4. Although they are not derived for specifically Michigan concrete, they may be used for practical purposes.

Table 4.4. Overstrength factors

Proposed overstrength factors	$f'_{ci_{measured}} / f'_{ci_{specified}}$	$f'_{c_{measured}} / f'_{c_{measured}}$
Michigan factors*	1.20	1.20
Rizkalla et al. (2013)	1.25	1.45
Rosa et al. (2007)	1.10	1.25
*Proposed in this thesis.		

The overstrength factors given in Table 4.4 was also provided in computer aided program *MDOTCamber* (Figure D.8).

4.6. Relationship between Calculated and Measured E_{ci}

Since the modulus of elasticity of concrete at release, E_{ci} , is one of the key parameters for accurately estimating camber at release as well as long-term camber, the relationship between the calculated modulus at release and the measured modulus was investigated. The calculated modulus was obtained using Eqns. 3-104, 3-106, 3-108 and 3-110 provided in the AASHTO LRFD Bridge Design Specifications (2020), ACI 318-19, ACI 363R-92 (1992), and fib MC 2010, respectively. Modulus of elasticity was calculated once using the specified concrete compressive strength, specified unit weight, and $K_1 = 1.0$ (Table 4.5), and another time using measured compressive strength (Table 4.6). The calculated modulus was compared to the measured modulus for several bridge projects. Modulus of elasticity tests were conducted in accordance with ASTM C469. The results are shown in Table 4.3 and 4.4. As can be seen, Eqns. 3-104, 3-106, 3-108 and 3-110 underestimate modulus of elasticity at release of prestressing force. When specified compressive strengths are used average ratios of measured over predicted modulus for AASHTO LRFD 2020, ACI 318-19, ACI 363R-92, and fib MC 2010 are 1.20, 1.24, 1.37, and 1.12 and COVs of 15%, 16%, 15%, and 15%, respectively. As can be seen, the fib MC 2010 model provides the most accurate and consistent estimations of modulus. When measured compressive strength are used, average ratios of measured over predicted modulus for AASHTO LRFD 2020, ACI 318-19, ACI

363R-92, and fib MC 2010 are 1.13, 1.14, 1.30, and 1.09 and COVs of 5%, 5%, 4%, and 5%, respectively. As can be seen, the fib MC 2010 model still provides the most accurate and consistent estimations of modulus. It should be noted that this conclusion is based on only a limited set of data for Michigan concretes. The proposed prediction methodology allows the use of any of these models for predicting modulus. The user of the proposed prediction tool is encouraged to evaluate each model. As a starting point, it is recommended that the AASHTO LRFD 2020 model be used since it is provided in the main body of AASHTO LRFD Bridge Design Specifications (2020) and since this model together with other models for the variation of modulus with time and creep and shrinkage models resulted in reasonable predictions of pre-erection and service camber and displacements when compared with measured values as will be demonstrated in the subsequent chapters. The use of measured compressive strength improved the accuracy and consistency of modulus of elasticity predictions. The specified overstrength factors for compressive strength can be used in the proposed prediction methodology to utilize this increase in modulus prediction accuracy.

Table 4.5. Relationship between calculated and measured modulus of elasticity at release, E_{ci} (calculated modulus is based on specified compressive strength)

Fabricator	Project	Beam ID	$E'_{ci, calculated}$ * (ksi)				$E'_{ci, measured}$ (ksi)	Ratio = $E'_{ci, measured}/E'_{ci, calculated}$			
			AASHTO (2020) [†]	ACI 318-19	ACI 363R-92	fib MC 2010 [†]		AASHTO (2020) [†]	ACI 318-19	ACI 363R-92	fib MC 2010 [†]
Peninsula Prestress	Concord Avenue	NA	4960	4821	4347	5280	4461	0.90	0.93	1.03	0.84
		NA	4960	4821	4347	5280	4682	0.94	0.97	1.08	0.89
	R04	NA	4960	4821	4347	5280	6533	1.32	1.36	1.50	1.24
		NA	4960	4821	4347	5280	6440	1.30	1.34	1.48	1.22
		NA	4960	4821	4347	5280	6346	1.28	1.32	1.46	1.20
	R04	N1	4960	4821	4347	5280	6272	1.26	1.30	1.44	1.19
	S02	O1	4484	4273	3967	4872	6443	1.44	1.51	1.62	1.32
	B02	P1	4960	4821	4347	5280	5320	1.07	1.10	1.22	1.01
	S07	NA	4960	4821	4347	5280	6377	1.29	1.32	1.47	1.21
*Using specified properties; [†] Aggregate factors, K_1 and α_E , were taken 1.0.							Avg.	1.20	1.24	1.37	1.12
							St. Dev	0.18	0.19	0.21	0.17
							COV (%)	15.3	15.7	15.1	15.0

Table 4.6. Relationship between calculated and measured modulus of elasticity at release, E_{ci} (calculated modulus is based on measured compressive strength)

Fabricator	Project	Beam ID	$E'_{ci\_calculated}$ * (ksi)				$E'_{ci\_measured}$ (ksi)	Ratio= $E'_{ci\_measured}/E'_{ci\_calculated}$			
			AASHTO (2020) [†]	ACI 318-19	ACI 363R-92	fib MC 2010 [†]		AASHTO (2020) [†]	ACI 318-19	ACI 363R-92	fib MC 2010 [†]
Peninsula Prestress	R04	NA	5790	5733	4980	5926	6533	1.13	1.14	1.31	1.10
		NA	5790	5733	4980	5926	6440	1.11	1.12	1.29	1.09
		NA	5790	5733	4980	5926	6346	1.10	1.11	1.27	1.07
	R04	N1	5343	5249	4644	5588	6272	1.17	1.19	1.35	1.12
	S02	O1	5315	5218	4622	5566	6443	1.21	1.23	1.39	1.16
	B02	P1	4991	4855	4371	5305	5320	1.07	1.10	1.22	1.00
	S07	NA	5844	5791	5020	5966	6377	1.09	1.10	1.27	1.07
*Using measured properties; [†] Aggregate factors, K_1 and α_E , were taken 1.0.							Avg.	1.13	1.14	1.30	1.09
							St. Dev	0.05	0.05	0.06	0.05
							COV (%)	4.5	4.6	4.4	4.5

4.7. Relationship between Measured and Predicted Variation of Modulus of Elasticity with Time

In addition to comparison of modulus of elasticity at release, E_{ci} , using measured and predicted values, the development of modulus of elasticity with time was also examined by conducting modulus of elasticity tests at various concrete ages and by comparing the measured moduli with the predicted ones using specified as well as measured compressive strengths. Results are presented in Figure 4.6. Figure 4.6 suggests that the use of measured compressive strengths resulted in closer agreements between measured and predicted time-dependent modulus. Figure 4.6 also suggests that the use of the fib MC 2010 model and the combination of AASHTO LRFD 2020 and ACI 209R-92 model resulted in more accurate predictions compared with the combination of ACI 363R-92 and ACI 209R-92 model. As noted earlier, the amount of long-term modulus data is too limited to draw any firm conclusions about the superiority of a given model for Michigan concretes. Since the proposed prediction methodology allows the use to select any combination of models the user is encouraged to explore this benefit to obtain a sense for the range of variability in the results for a given project. However, the combination of the AASHTO LRFD 2020 and ACI 209R-92 models can be used as the starting point for predicting the modulus of elasticity with time.

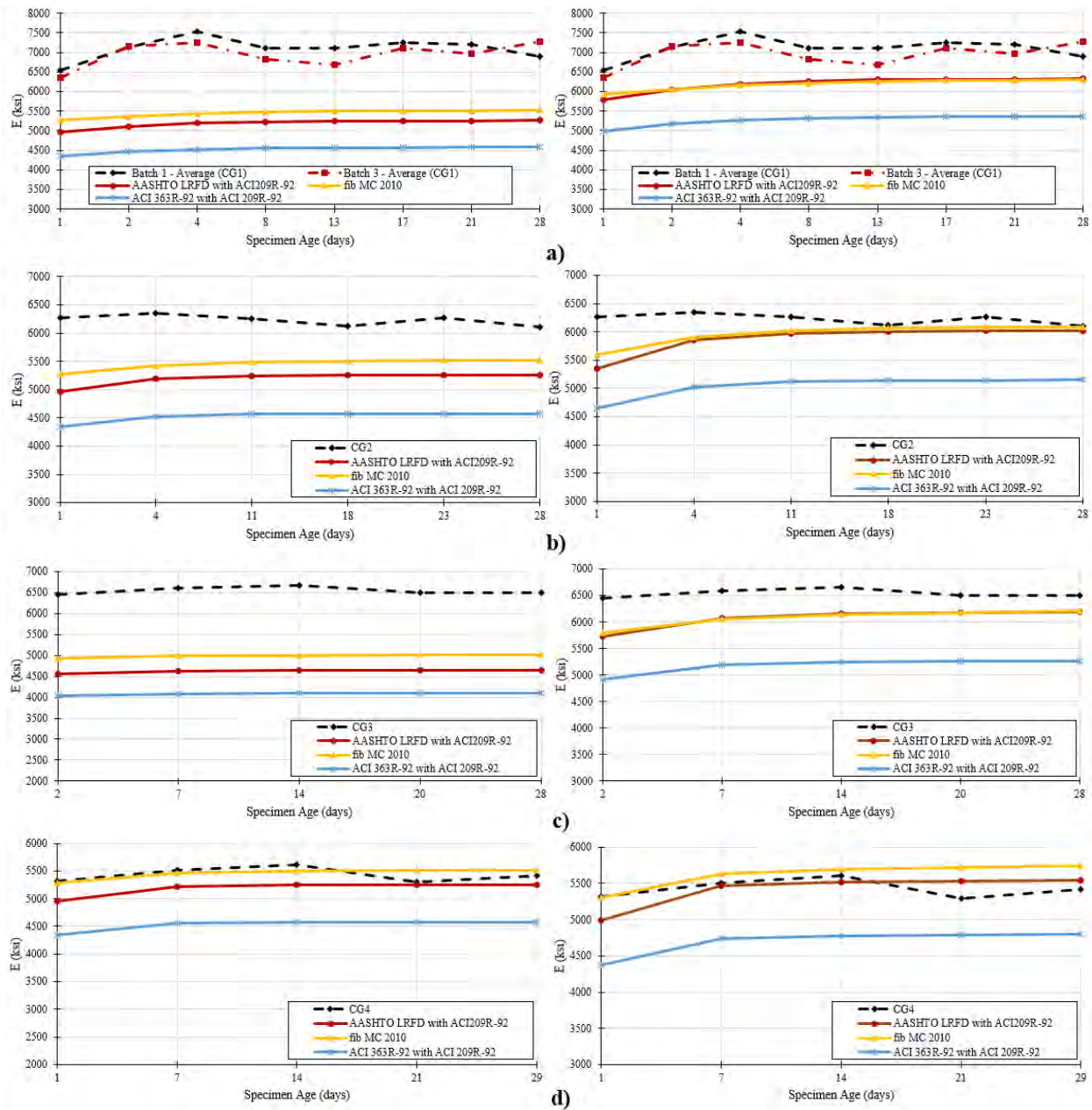


Figure 4.6. Development of modulus of elasticity with time using specified (left) and measured (right) properties for: a) R04 (CG1); b) R04 (CG2); c) S02 (CG3); d) B02 (CG4) projects

4.8. Relationship between Specified and Measured Concrete Unit Weight

The relationship between the specified and measured unit weight was investigated for the beams fabricated by Peninsula Prestress. Since concrete unit weight is an important factor when estimating the self-weight of the beam and consequently the displacement due to self-weight, and

calculation of modulus of elasticity for some models such as AASHTO LRFD 2020 and ACI 318-19 and ACI 363-92, the quantification of any variation in this parameter is of interest.

Table 4.7. Comparison of specified and measured concrete unit weight

Fabricator	Project Name	# of Beams	$w_{specified}$ (pcf)	$w_{measured}$ (pcf)	$R = w_{measured}/w_{specified}$
Peninsula Prestress	Livernois	11	145	148.64	1.03
	Clark Ave.	4	145	149.24	1.03
	S05	11	145	145.11	1.00
	S11	10	145	145.33	1.00
	M311	4	145	145.85	1.01
	M89	1	145	145.12	1.00
	M5	1	145	145.42	1.00
	M57	5	145	144.80	1.00
	Average				1.01
St. Dev.				0.01	
COV (%)				1.18	

The results shown in Table 4.7 suggest that there is not much variation between the specified and measured unit weight. The average ratio between measured and specified unit weight is 1.01 and COV is 1.18%.

4.9. Relationship between Specified and Measured Prestressing Force

One of the parameters that affect the magnitude of camber is the magnitude of the prestressing force. Therefore, using the correct prestressing force when estimating short-term or long-term camber is paramount. The relationship between the measured and specified prestressing force was investigated to determine the degree of variability. The results shown in Table 4.8 suggest that there is not much variation between the specified and measured prestressing force. The average ratio between measured and specified prestressing force is 1.02 and the COV is 0.87%.

Table 4.8. Comparison of specified and measured prestressing force at release

Fabricator	Project Name	# of Beams	$P_{i\text{specified}}$ (lbs)	$P_{i\text{measured}}$ (lbs)	$R = P_{i\text{measured}}/P_{i\text{specified}}$
Peninsula Prestress	M89	6	44000	45772	1.03
	M57	9	44000	45400	1.03
	M311	6	44000	44600	1.01
	S05	11	44000	45156	1.03
	S11	10	44000	45086	1.02
Avg.					1.02
Std.					0.01
COV (%)					0.87

4.10. Relationship between Specified and Measured Beam Length

Beam length is one of the key parameters when estimating displacements and camber because a small variation in it causes large changes in displacements and camber. For example, displacements due to beam self-weight are a function of the beam length raised to the power of four. Similarly, camber due to prestress is a function of beam length raised to the power of two. The relationship between the specified beam length and supplied one was investigated for four Michigan projects whose beams were fabricated by Peninsula Prestress. The results of such comparison are shown in Table 4.9. As can be seen, there is negligible variation between the specified and measured beam length. Therefore, beam length can be eliminated as a source of uncertainty.

Table 4.9. Comparison of specified and measured beam length

Fabricator	Project Name	# of Beams	$L_{\text{specified}}$ (in.)	L_{measured} (in.)	$R = L_{\text{measured}}/L_{\text{specified}}$
Peninsula Prestress	M89	5	355	355.07	1.00
	M5	6	1381	1380.87	1.00
	M57	8	1664.63	1665	1.00
	Shepherd Rd. 1-3	1	671	672	1.00
	Shepherd Rd. 2	1	750	750	1.00
Average					1.00
St. Dev.					0.00
COV (%)					0.06

4.11. Summary of Relationships between Measured and Specified/Calculated Properties

Table 4.10 provides a summary of the relationships between measured and specified/calculated properties using various statistical indicators. The average ratio and COV of measured over specified concrete compressive strength at prestress release and 28 days was 1.21 and 1.21, and 14% and 11%, respectively. Therefore, the algorithm based on Approach No. 6 provides the user the option to use Michigan Overstrength Factor of 1.2 for both f'_{ci} and f'_c . The average ratio and COV of measured over calculated modulus of elasticity at prestress release based on AASHTO LRFD Specifications (2020) are 1.20 and 15%, respectively, when the specified compressive strength was used to calculate the modulus. When measured properties were used, the average ratio and COV of measured over calculated modulus of elasticity at prestressed release based on AASHTO LRFD Specifications (2020) are 1.13 and 4.5%, respectively. Finally, as previously indicated there was negligible variation between measured and specified unit weight, prestressing force, and beam length.

Table 4.10. Summarized results for the relationship between measured and specified parameters

Statistical parameters	$\frac{f'_{ci\_measured}}{f'_{ci\_specified}}$	$\frac{f'_c\_measured}{f'_c\_specified}$	$\frac{E_{ci\_measured}}{E_{ci\_calculated}^*}$	$\frac{E_{ci\_measured}}{E_{ci\_calculated}^{**}}$	$\frac{w_{measured}}{w_{specified}}$	$\frac{P_{i\_measured}}{P_{i\_specified}}$	$\frac{L_{measured}}{L_{specified}}$
Min.	0.94	1.01	0.90	1.07	1.00	1.01	1.00
Max.	1.78	1.90	1.44	1.21	1.03	1.03	1.00
Avg.	1.21	1.21	1.20	1.13	1.01	1.02	1.00
COV. (%)	14	11	15	4.5	1.18	0.87	0.06
# of tests	299	234	9	7	47	42	21
*Based on AASHTO LRFD (2020) with specified concrete compressive strength.							
**Based on AASHTO LRFD (2020) with measured concrete compressive strength.							

4.12. Solar Radiation Study

To quantify the influence of solar radiation induced temperature gradient on beam camber, several measurements were taken for the beams fabricated by PSI Windsor on April 27, 2019 between 8:30 am and 1:30 pm in Windsor, Canada. Three measurements were taken for each beam: the first at 8:30-9:00 am, the second at 11:00-11:30 am, and the third at 1:00-1:30 pm. The weather

during the day was mostly sunny and the temperature varied from 38 °F to 51 °F. All four girders were directly exposed to sunlight throughout the time measurements were taken. The measured data is illustrated graphically in Figure 4.7. Camber varied as much as 0.22 in. due to the influence of daily temperature gradient. This was a smaller variation than that reported by Hinkle (2006) in Virginia who reported that camber varied as much as 0.5 in. due to solar radiation induced temperature gradient. However, Hinkle (2006) conducted his measurements in August 25, in Virginia, between 7:30 am and 1:30 pm. The recorded temperatures varied from 82 °F to 90 °F. The differences between the camber variation that was measured in Windsor, Canada and that reported by Hinkle (2006) in Virginia are attributed to differences in climate and season although both Michigan and Virginia fall in solar radiation zone III (Windsor, Canada is assumed to be in the same solar radiation zone due to its proximity to Michigan).

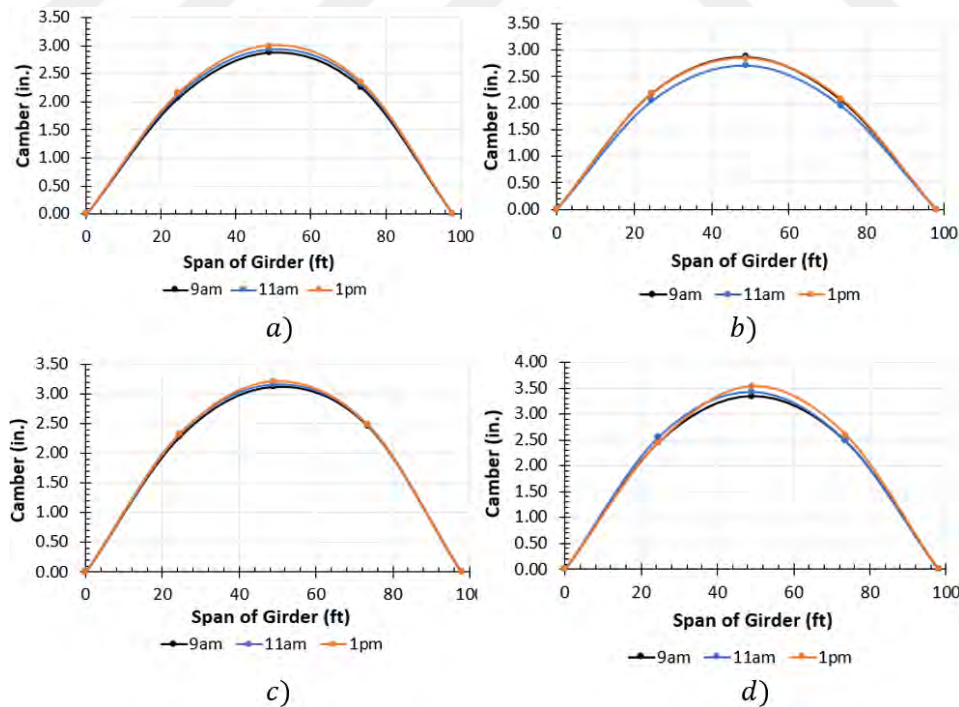


Figure 4.7. Variation in camber due to radiation induced thermal gradient for: a) Girder G2-6, b) Girder G2-7, c) Girder G2-8, d) Girder G2-9

Moreover, another trip was conducted to quantify the effect of temperature gradient on beam camber and displacements. To do so, a set of measurements throughout the day were collected for the box beams fabricated by Peninsula Prestress plant in Grand Rapids, Michigan on March 10, 2020. However, the change in camber due to radiation induced thermal gradient could not be observed clearly due to inappropriate weather conditions. The outside temperature fluctuated between 55 °F and 60 °F, and the weather was cloudy. As seen in Figure 4.8, there is no clear trend in the camber due to temperature gradient.

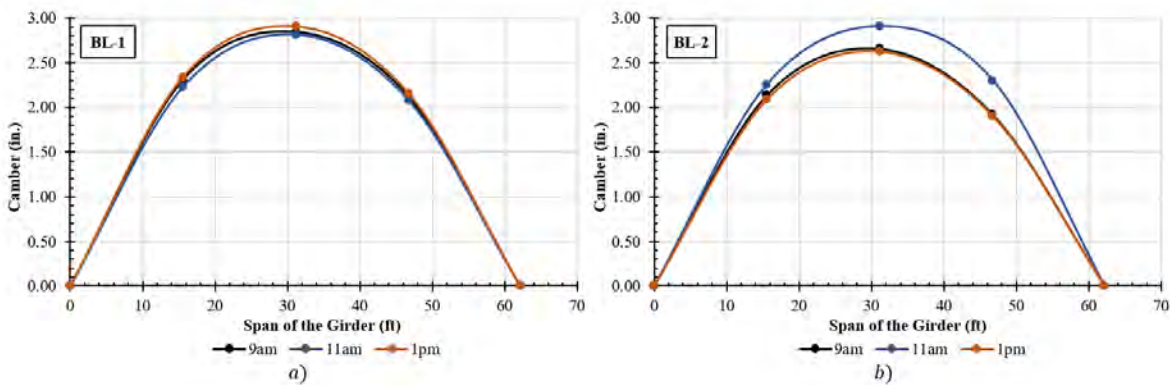


Figure 4.8. Variation in camber due to radiation induced thermal gradient for: a) Girder BL-1 and b) Girder BL-2

Finally, another set of measurements to explore the radiation induced change in camber were collected from the box beam located near Wayne State University structure's lab on June 30, 2020. The beams were cast by Peninsula Prestress fabricator and features 57 ft. span length. There were two beams in total, however, due to tight storage conditions, measurements for only one beam could be taken. Like in previous studies, three sets of measurements starting early in the morning at girder ends, $\frac{1}{4}$ and $\frac{3}{4}$ of the span length, and midspan were collected. Although the weather temperature varied between 75 °F and 90 °F, no significant change in camber was observed, as seen in Figure 4.9a. The reasons why camber growth could not be observed may be explained in two reasons. The first is that the support conditions of the beam were not adequate.

The wooden supports were not placed to girder ends, but approximately 15 ft. from girder ends. Indeed, there were three wooden blocks supporting the girder, and due to uneven ground condition, all there were touching the girder. Therefore, it is not fair to say that the girder behaves as a simply supported beam. In addition to this, just near to the beam where measurements were taken, there was a crane which applied a force at the tip of the beam, as shown Figure 4.9b. This could be another factor that may affect the analysis results.

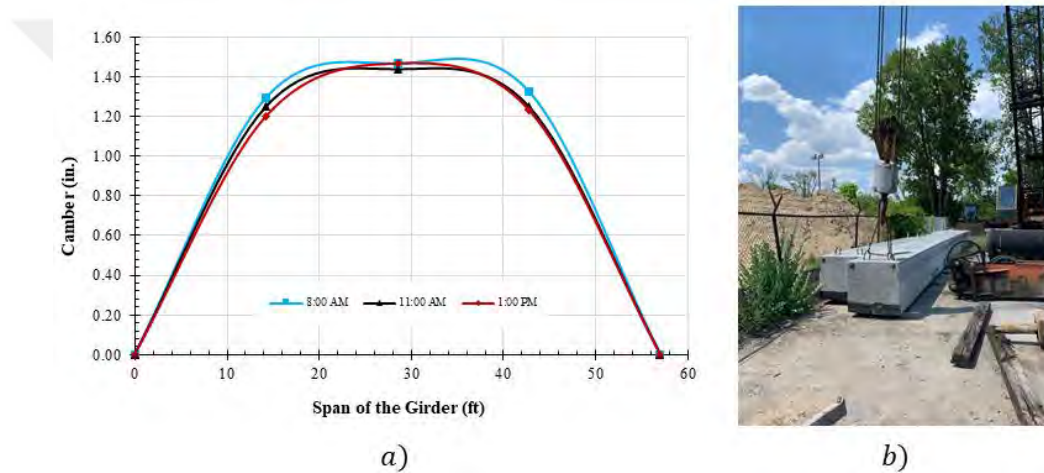


Figure 4.9. a) Variation in camber due to radiation induced thermal gradient, b) A view for the beams located near Wayne State University structure's lab

4.13. Data Collection Guidelines

There are many factors that affect short-term and long-term camber, and each factor is associated with some degree of uncertainty. To minimize uncertainties related to the influence of radiation induced temperature gradient it is recommended that camber measurements are collected at a specific time frame during the day, such as early in the morning and this time should be indicated in the survey report. Additionally, to further quantify the influence of solar radiation induced temperature gradient it would be useful to collect multiple readings during the same day at various times in the day such as early morning, mid-morning, noon, and afternoon. In fact, the weather conditions should be considered while performing the solar radiation study: The weather

temperature should rise up as the time passes and girders should be positioned such a way that they can be exposed to direct sun light, if possible. Additionally, the storage and support conditions for the beams should be indicated in the survey report to properly quantify the influence of the self-weight of the beam on the reported beam elevations throughout the span. Furthermore, the instrument with which camber is measured should be indicated in the survey report to ensure that reliable techniques are used to measure camber. Also, to investigate the relationship between the specified and measured concrete compressive strength, it is recommended that such data be collected in as many projects as possible including past projects as this information is typically reported by the precast fabricators.

CHAPTER 5: Evaluation of Factors that Affect Prestressed Concrete Beam Camber and Deflection

5.1. Introduction

Factors known to affect camber at release of prestressing force are unit weight of concrete, member geometry (cross-section and beam length), strand layout and magnitude of the prestressing force, modulus of elasticity at prestress release, transfer length, debonding, intended or unintended cracking, support conditions during storage, and temperature gradients. A sensitivity analysis was conducted to quantify the influence of several of these factors on camber at release and long-term as shown in as shown in were AASHTO LRFD 2020 for the prediction of modulus of elasticity at release and 28 days; ACI 209R-92 with back-calculated parameters for the variation of modulus of elasticity with time; and AASHTO LRFD 2020 Body provision for creep and shrinkage calculations. During the analysis, 51 sections were considered, and specified concrete properties with no overstrength factors were utilized. The parameters that were considered in the sensitivity analysis include the length of the beam, the unit weight of concrete, the magnitude of the prestressing force, modulus of elasticity at prestress release, and the time when initial camber is measured. For pre-erection camber, additional parameters such as the creep and shrinkage model, the model to predict modulus of elasticity at prestress release and 28 days as well as its variation with time, and overstrength factors were considered to quantify their influence on pre-erection camber.

5.2. Sensitivity Analysis

The results of the sensitivity analysis are shown in Table 5.1 and Table 5.2. The analysis was conducted using the approach presented in Approach No. 6 for each project using specified properties and then the considered parameters were varied by +/- 10%. The results in Table 5.1

and Table 5.2 show the average, standard deviation, covariance as well as the lower and upper bounds in terms of % change in net deflection (i.e. camber) for all projects considered.

A +/- 10% change in the length of the beam resulted in a change in camber at release that varied from -18.6% to 17.1%. The average change in camber at release for a +/-10% change in beam length was 8.6% and -12.1% with 61% and -29% covariance, respectively. A similar influence was observed for pre-erection camber. It is interesting to note the range of the percent change in net camber due a fixed percent change in the length of the beam. This range emphasized the need to conduct the sensitivity analysis for various projects rather than draw conclusions based on conducting a sensitivity analysis for a single project. While the results suggest that a 10% change in beam length can cause as high as a 17.1% change in camber, the results presented in section 4.10 suggest that the variation in beam length is minimal and this parameter does not constitute a source of high uncertainty. Additionally, while beam length is expected to have an exponential effect on either prestressed induced camber or gravity induced downward displacement, the net effects is as shown by the lower and upper bounds as well as the average change and corresponding standard deviation and covariance. This is due to the competing effects of prestressing force and gravity as well as other factors such as support conditions during storage.

A +/- 10% change in the specified compressive strength at release resulted in a change in camber at release that varied from -4.9% to 5.6% and a change in pre-erection camber that varied from -6.9% to 8.2%. The average change in camber at release for a +/-10% change in $f'_{ci_{beam}}$ was -4.4% and 5% with -4% and 5% covariance, respectively. While the compressive strength itself does not influence in service behavior during which stresses are within the linear elastic range of the stress strain curve for concrete, compressive strength is typically used as an indicator to characterize the class of concrete being used including its stiffness (modulus of elasticity). Because

the compressive strength is used in the empirical equations used to calculate modulus, it has a marked influence on initial and pre-erection camber. Its higher influence on the pre-erection camber is explained by the fact that the compressive strength is a parameter in the creep model provided in the AASHTO LFRD Bridge Design Specifications (2020).

A +/- 10% change in the magnitude of the prestressing force at jacking resulted in a change in camber at release that varied from -20.7% to 20.7%. The average change in camber at release for a +/-10% change in $P_{jacking}$ was 15.1% and -15.2% with 14% and -13% covariance, respectively. A similar influence is observed for pre-erection camber. As can be seen, the influence of a change in the magnitude of the prestressing force, similar to that of a change in beam length, has a significant influence on the magnitude of calculated camber. The maximum influence of a change in the magnitude of the prestressing force is slightly higher than that of a change in beam length (20.7% versus 17.1%). However, the results presented in section 4.10 indicate that precast fabricators have good control over the magnitude of the applied prestressing force, therefore, this parameter does not constitute a source of high uncertainty.

A +/- 10% change in the magnitude of the unit weight of concrete resulted in a change in camber at release that varied from -10.8% to 10.8%, suggesting this parameter also has a significant influence. The average change in camber at release for a +/-10% change in w was -5.2% and 5.1% with -38% and 41% covariance, respectively. A similar influence was observed for pre-erection camber. However, the results presented in section 4.8 suggest that precast fabricators have good control over the supplied unit weight of concrete used for the prestressed concrete beams and that the measured unit weight matches well with the specified unit weight.

A +/- 10% change in the modulus of elasticity of beam concrete at release resulted in a change in camber at release that varied from -9.8% to 10.6%. The average change in camber at

release for a +/-10% change in E_{ci} was -8.6% and 10.3% with -4% and 2%, respectively. A similar change was observed for pre-erection camber. This suggests that a certain percent change in the initial modulus results in a comparable change in camber at release. Although the influence of the modulus of elasticity of concrete does not appear to be as high as that of beam length or prestressing force, this parameter constitutes a high source of uncertainty, as highlighted in section 4.6, compared to the other three parameters discussed above. This is due to the fact that the equation used to determine the modulus of elasticity was obtained using curve fitting techniques and the scatter in data is large. Naturally, the influence of the initial and 28 day modulus on beam camber and displacements was stronger than that of the initial beam concrete compressive strength. This is because the former has a direct influence on camber and displacements, whereas the latter only influences camber and displacements through the calculated modulus (which is a function of $\sqrt{f'_{ci}}$).

The influence of the support locations during beam storage in the precast facility on initial and pre-erection camber was investigated by varying the location of these supports by +/- 10%. A +/- 10% variation in support locations resulted in -1.4% to 0.8% change in camber at release and -1.0% to 0.9% change in pre-erection camber. The average change in camber at release for a +/- 10% change in $L_{overhang}$ was 0.4% and -0.6% with 42% and -45% covariance, respectively. These results suggest that support location influences pre-erection camber to a degree that may be worth considering in the prediction methodology. As a result, the prediction framework provided as part of this report includes provisions for accounting for the support locations during storage.

A +/- 10% change in transfer length resulted in a +/-1.4% change in initial camber and +/- 0.9% change in pre-erection camber, suggesting that the influence transfer length may also be worth considering. The average change in camber at release for a +/-10% change in $L_{transfer}$ was

-0.1% and 0.1% with -243% and 312% covariance, respectively. The triple digit covariance percentage is explained that although the standard deviation for a +/-10% change in $L_{transfer}$ was about 0.3, due to division of it to a very small number, covariance was obtained very high.

A similar observation was made for the influence of debonded length for beams that featured strands that were debonded from concrete for a certain length at the ends of the beams to control the magnitude of tensile stresses at the ends of the beam. A +/- 10% change in debonded length resulted in a +/-1.4% change in initial camber and a +/-1.8 change in and pre-erection camber, suggesting again that debonded length may be worth considering in the prediction methodology. The average change in camber at release for a +/-10% change in $L_{debonded}$ was -0.4% and 0.3% with -124% and 136% covariance, respectively.

A +/- 10% change in the location of the harping points resulted in -3.0% to 2.7% change in initial camber and -3.0% to 2.7% change in pre-erection camber suggesting that the location of the harping points has a marked influence on pre-erection camber and should be accounted for in the prediction methodology. The average change in camber at release for a +/-10% change in $L_{harping}$ was -1.9% and 1.7% with -37% and 39%, respectively.

The proposed prediction methodology (Approach No. 6) accounts of support locations during storage, transfer length, debonded length, and location of harping point thus providing the user a tool with great flexibility and which accounts for a variety of scenarios and designs.

Table 5.1. Influence of various parameters on camber at release

Camber at Release Sensitivity Analysis					
Parameter varied	% Change in net camber/displacement				
	Lower bound % change for net camber	Upper bound % change for net camber	Average	St. Dev.	COV (%)
Baseline (Approach No. 6)	-	-	-	-	-
$L_G +10\%$	-4.8	17.1	8.6	5.2	61
$L_G -10\%$	-18.6	-4.1	-12.1	3.4	-29
$f'_{ci_{beam}} +10\%$	-4.9	-4.0	-4.4	0.2	-4
$f'_{ci_{beam}} -10\%$	4.3	5.6	5.0	0.3	5
$P_{jacking} +10\%$	11.4	20.7	15.1	2.1	14
$P_{jacking} -10\%$	-20.7	-12.2	-15.2	2.0	-13
$w +10\%$	-10.8	-2.4	-5.2	2.0	-38
$w -10\%$	1.4	10.8	5.1	2.1	41
$E_{ci} +10\%$	-9.8	-8.4	-8.6	0.3	-4
$E_{ci} -10\%$	9.8	10.6	10.3	0.2	2
$L_{overhang} +10\%$	0.0	0.8	0.4	0.2	42
$L_{overhang} -10\%$	-1.4	-0.3	-0.6	0.3	-45
$L_{transfer} +10\%$	-1.4	0.0	-0.1	0.3	-243
$L_{transfer} -10\%$	0.0	1.4	0.1	0.3	312
$L_{debonded} +10\%$	-1.4	0.0	-0.4	0.5	-124
$L_{debonded} -10\%$	0.0	1.4	0.3	0.4	136
$L_{harping} +10\%$	-3.0	-0.8	-1.9	0.7	-37
$L_{harping} -10\%$	0.5	2.7	1.7	0.6	39

L_G = Beam length, $f'_{ci_{beam}}$ = compressive strength of beam concrete at release, $P_{jacking}$ = prestressing force at jacking, w = unit weight, E_{ci} = modulus of elasticity at release, $L_{overhang}$ = beam overhang length, $L_{transfer}$ = transfer length, $L_{debonded}$ = debonded length, $L_{harping}$ = location of harping point measured from girder end

Table 5.2. Influence of various parameters on pre-erection camber

Pre-erection Camber Sensitivity Analysis					
Parameter varied	% Change in net camber/displacement				
	Lower bound % change for net camber	Upper bound % change for net camber	Average	St. Dev.	COV (%)
Baseline (Approach No. 6)	-	-	-	-	-
$L_G +10\%$	-7.7	18.2	8.1	6.1	75
$L_G -10\%$	-17.3	-2.6	-11.5	3.5	-30
$f'_{ci_{beam}} +10\%$	-6.9	-5.4	-5.8	0.4	-7
$f'_{ci_{beam}} -10\%$	6.6	8.2	7.0	0.5	7
$P_{jacking} +10\%$	12.7	21.8	15.4	2.1	14
$P_{jacking} -10\%$	-22.0	-11.5	-15.3	2.4	-15
$w +10\%$	-12.1	-1.8	-5.4	2.4	-44
$w -10\%$	2.7	11.9	5.6	2.1	39
$E_{ci} +10\%$	-9.7	-7.7	-9.0	0.4	-4
$E_{ci} -10\%$	10.7	11.8	11.0	0.3	3
$L_{overhang} +10\%$	0.0	0.9	0.5	0.2	42
$L_{overhang} -10\%$	-1.0	0.0	-0.5	0.2	-45
$L_{transfer} +10\%$	-0.9	0.0	-0.1	0.2	-154
$L_{transfer} -10\%$	0.0	0.9	0.1	0.2	218
$L_{debonded} +10\%$	-1.8	0.0	-0.4	0.5	-126
$L_{debonded} -10\%$	0.0	1.8	0.4	0.5	138
$L_{harping} +10\%$	-3.1	-0.6	-1.9	0.8	-40
$L_{harping} -10\%$	0.6	2.7	1.7	0.6	37
L_G = Beam length, $f'_{ci_{beam}}$ = compressive strength of beam concrete at release, $P_{jacking}$ = prestressing force at jacking, w = unit weight, E_{ci} = modulus of elasticity at release, $L_{overhang}$ = beam overhang length, $L_{transfer}$ = transfer length, $L_{debonded}$ = debonded length, $L_{harping}$ = location of harping point measured from girder end					

5.2.1. Influence of Creep and Shrinkage Model on Pre-erection Camber

The influence of the selected creep and shrinkage model for beam concrete on pre-erection camber was investigated by computing pre-erection camber for the first 28 days for the prestressed concrete beams used in the S11 project. In this study, AASHTO LRFD 2020 model was used for the modulus of elasticity prediction, and ACI 209R-92 model with back-calculated parameters was utilized for its variation with respect to time. Analysis was conducted using specified properties with no overstrength factors and 51 sections along the half-span. The results are shown in Figure

5.1. Predictions based on Bazant-Baweja B3 model and GL 2000 model were significantly higher than measured pre-erection camber values as well as predictions based on other models and are therefore not included in Figure 5.1. Figure 5.1 suggests that the selection of the creep and shrinkage model for beam concrete has a marked influence on the prediction of pre-erection camber development. The proposed prediction methodology (Approach No. 6) provides the user the ability to choose one out of eight models. Since most predictions presented in this report were based on the AASHTO LRFD 2020 creep and shrinkage models provided in the body of AASHTO LRFD 2020, and since these predictions matched well with measured pre-erection camber, it is recommended that these models can be used as a starting point. Other models may be used to quantify the expected range of camber predictions.

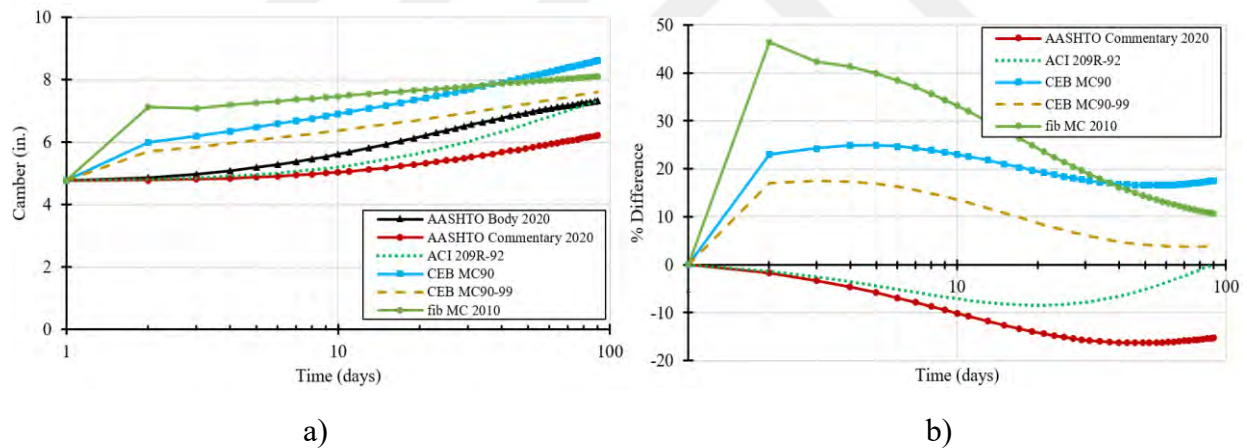


Figure 5.1. a) Variation of pre-erection camber for S11 project using different creep and shrinkage models, and b) % Difference in the pre-erection camber for S11 project using different creep and shrinkage models with respect to AASHTO LRFD 2020 Body benchmark model

5.2.2. Influence of Model to Predict Modulus of Elasticity at Prestress Release and 28 Days

The influence of the model to predict modulus of elasticity at prestress release and 28 days on pre-erection camber was evaluated by running the proposed prediction methodology four times for the S11 project: one time with the benchmark model (AASHTO LRFD 2020) and three other times using ACI 318-19 model, ACI 363R-92 model, and fib MC 2010 model, respectively. When

using the AASHTO LRFD 2020 model, the aggregate factor, K_1 , was taken equal to 1.0 and concrete unit weight, w_c , was calculated using the compressive strength dependent formula, as provided in Eqns. 3-104 and 3-105. Additionally, the ACI 209R-92 model was selected to capture the variation of modulus with time, no overstrength factors were used, and the AASHTO LRFD 2020 model for creep and shrinkage were used as part of the benchmark model. When using the fib MC 2010 model, the aggregate type was assumed to be quartzite. The number of beam sections along half the span was selected as 51, and the proposed time step generation method was used. Figure 5.2a shows a comparison of predicted moduli of elasticity at prestress release and 28 days using each model and, Figure 5.2b shows the difference in pre-erection camber magnitude when each model is considered. Figure 5.2b suggests that the selection of the model for modulus of elasticity at prestress release and 28 days has a noteworthy influence on pre-erection camber with the ACI 363R-92 model resulting in differences up to 14% and ACI 318-19 resulting in differences up to 6%. Since the benchmark model used in the evaluations presented in this report resulted in reasonable predictions of pre-erection camber when compared to measured values, it is recommended that the AASHTO LRFD 2020 model is used for prediction modulus of elasticity. Investigation of other models may be used to determine the limits of variability for pre-erection camber.

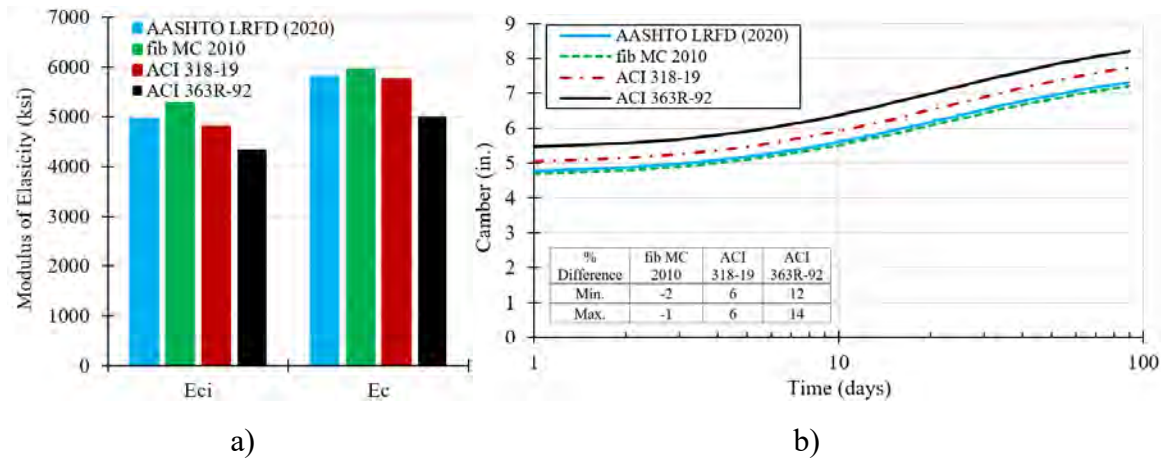


Figure 5.2. a) Comparison of calculated Modulus of Elasticity at Prestress Release and 28 days, and b) Influence of Model to Predict Modulus of Elasticity at Prestress Release and 28 Days for S11 project

5.2.3. Influence of the Model for Capturing the Variation of Modulus with Time

The influence of the model to capture the variation of modulus with time was selected by running the algorithm twice; once based on the benchmark model (i.e. ACI 209R-92), and another time using the fib MC 2010 model. When using the benchmark model, the AASHTO LRFD 2020 model for creep and shrinkage as well as prediction of modulus at prestress release and 28 days were used. The number of beam section along half the span was selected as 51 and the proposed time step generation method was used. The results are shown in Figure 5.3. Figure 5.3a shows the prediction of modulus as a function of time and suggests that there are some minor differences between the two models. Figure 5.3b shows that camber predictions based on both models are almost identical, therefore the selection of one model or the other is inconsequential. The % difference in pre-erection camber predictions is less than 0.1%.

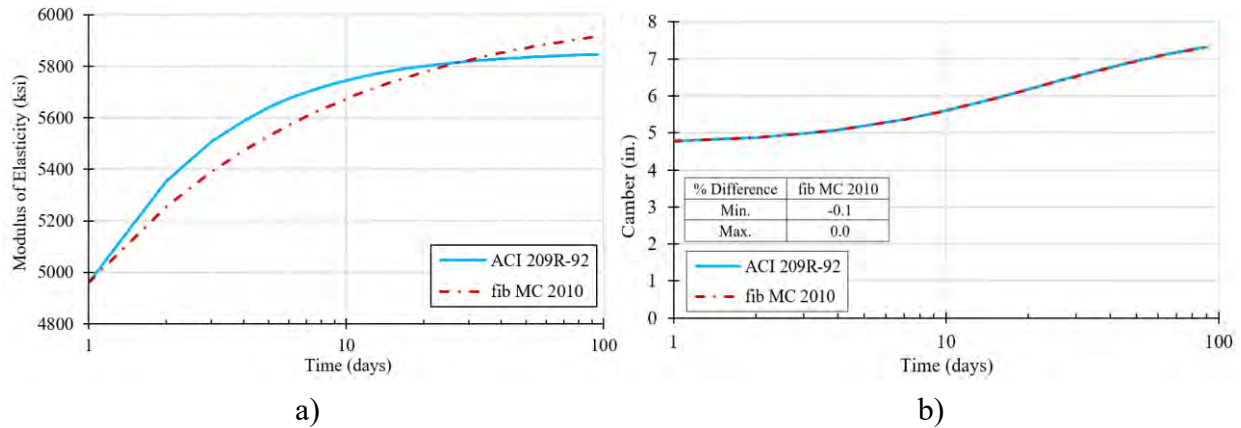


Figure 5.3. a) Comparison of functions for predicting the development of modulus of elasticity with respect to time, and b) influence of considered time dependent modulus functions on camber for S11 project

5.2.4. Influence of Overstrength Factors

The influence of overstrength factors for beam concrete compressive strength at prestress release and 28 days was investigated by running the algorithm once without overstrength factors and another time with Michigan based overstrength factors (Eqns. 4-1 and 4-2). It should be noted that the use of overstrength factors for f'_{ci} affects the calculation of modulus at prestress release, E_{ci} , which affects the calculation of both camber at release and pre-erection camber. Additionally, the use of overstrength factors affect both f'_{ci} and f'_c , which in turn affect moduli of elasticity at prestress release and 28 days. These moduli of elasticity at these two times serve as anchor points for the model that predicts the development of modulus with time. The benchmark model was based on no overstrength factors, AASHTO LRFD 2020 model for modulus at release and 28 days as well as creep and shrinkage, and the ACI 209R-92 model for capturing the variation of modulus with time. The only change in the second analysis was the inclusion of the overstrength factors for f'_{ci} and f'_c . The number of beam sections along half the span was selected as 51 and the proposed time step generation method was used. The results are shown in Figure 5.4. The inclusion of overstrength factors does have a marked effect on pre-erection camber predictions with differences between the two models being up to 10% as shown in Figure 5.4b. As expected, the inclusion of

overstrength factors results in lower camber predictions. It will be demonstrated in sections 8.2 and 8.3 later that the inclusion of overstrength factors results in more consistent results.

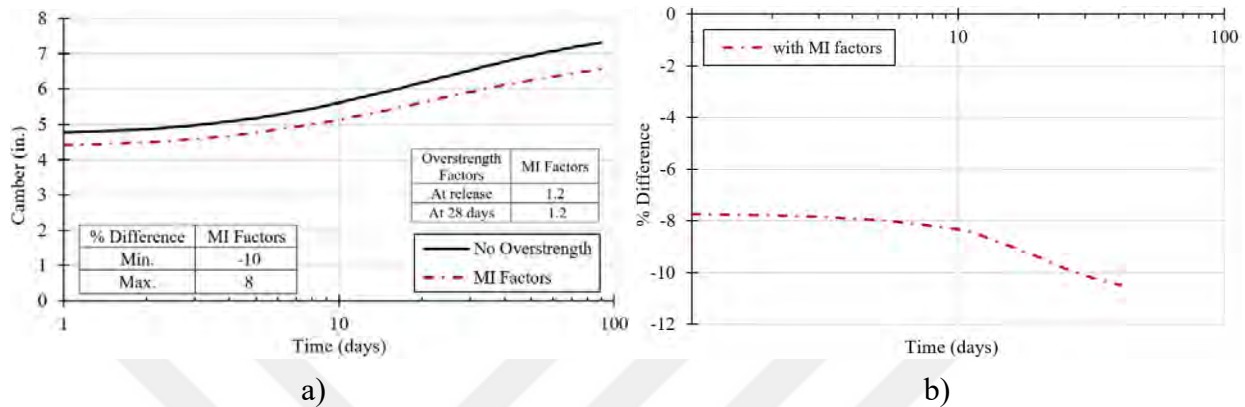


Figure 5.4. a) Influence of inclusion of overstrength factors on camber for S11 project, and b) % Difference in camber with respect to no overstrength factors

5.2.5. Influence of Time when Camber at Release is Measured

When camber at release is calculated, time-dependent effects are typically ignored because by definition the term camber at release means the camber induced in the prestressed concrete beam immediately after the detensioning of the strands. According to MDOT's Special Provision for Structure Survey during Construction, for projects with new prestressed concrete beam superstructures, the release date of the prestressing force and the observed camber at midspan for each beam should be provided to the engineer no more than 7 calendar days after releasing the prestressing force. The effect of time on camber at release was investigated by predicting camber for up to 10 days after the release of the prestressing force for four projects: M5, M89, Tiffin Street Overpass, and S11. This investigation was conducted by using the benchmark model set described in Figure 3.17 except that no overstrength factors were attributed. The results are shown in Figure 5.5. Detensioning time was assumed to be 1 day after placement of concrete. The change in percentage between the predicted initial camber at 1 day and 10 days varies from 17% to 26% introducing yet another source of uncertainty when comparing predicted and measured camber at

release values. This is not an issue when the proposed prediction methodology is used because it offers the ability to predict camber at any time, however, traditional techniques such as classical equations for prediction camber at release and the multiplier method for predicting long-term camber cannot capture these nuances in camber variation. In any case, this topic was investigated to quantify the variation in camber during the first 7 days so that the engineer can have a sense for the degree of this variation.

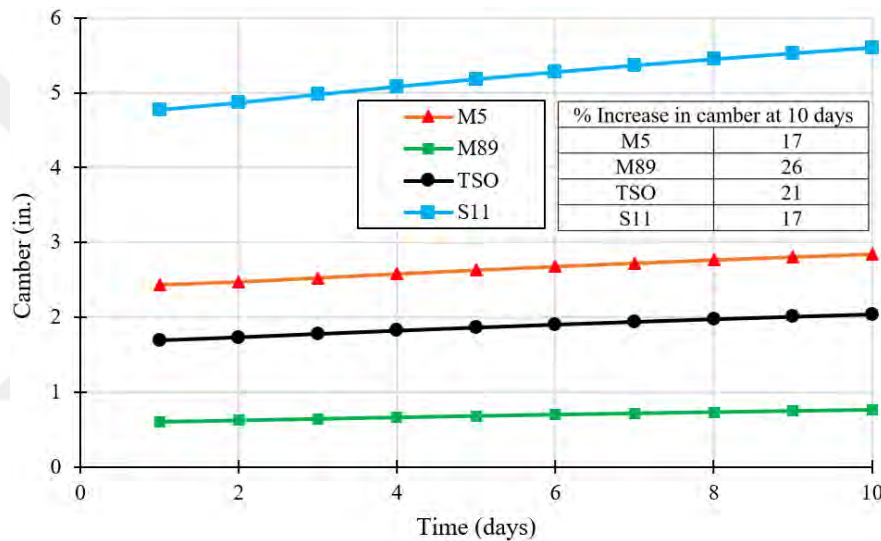


Figure 5.5. Variation of predicted camber during the first 10 days after beam fabrication

5.2.6. Slab and Screed Guidance

The proposed methodology provides camber data at various points along the span of the beam so that this information can be used to plot the full camber profile for the beam. The cambered beam profile can then be used to determine slab and screed elevations in accordance with MDOT Slab and Screed Guidance document. The proposed methodology may be cited as the tool to obtain beam deflection at various stages. Currently, in the MDOT Slab and Screed Guidance document the following stages of deflection are referenced: 1) Deflection of the beam due to self-weight (pre-erection camber), 2) Deflection of the beam due to the weight of forms and rebar, 3) Deflection of the beam due to the weight of deck concrete, and 4) Deflection of the beam due to

the weight of sidewalk or barrier. The proposed methodology provides beam camber data for each stage. This data can be downloaded so that beam profiles for each stage can be plotted and slab and screed elevations can be set accordingly. An example is provided in Figure 5.6, which shows the deflected shape of the beam at various stages. In this example, the following loads were used: 10 psf for formwork, 10 psf for reinforcement, 145 pcf for plain concrete, and 150 pcf for reinforced concrete, and 5 kips for midspan diaphragm. This is consistent with Michigan Design Manual Bridge Design - Chapter 7: LRFD Section 7.02.22.

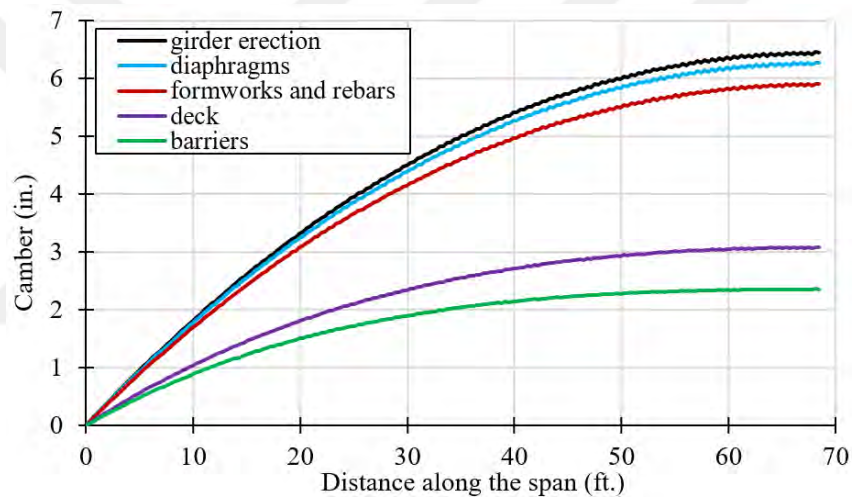


Figure 5.6. Illustration of various stages of deflection during beam erection and deck placement for the S11 project

5.3. Summary and Conclusions

While it is determined that the unit weight of concrete, w , the magnitude of the prestressing force at jacking, $P_{jacking}$, and beam length, L_G , all have a significant influence on pre-erection beam camber, it is determined that these parameters do not vary significantly from specified values and therefore do not represent a significant source of uncertainty. Modulus of elasticity at release, E_{ci} , had a proportional influence on pre-erection beam camber. Similarly, beam compressive strength at release, f'_{ci} , also had a close to proportional influence on pre-erection camber, although this influence was quantified through the use of compressive strength dependent equation for

modulus of elasticity. Transfer length, $L_{transfer}$, debonded length, $L_{debonded}$, support conditions during storage, $L_{overhang}$, and location of harping point, $L_{harping}$, influence pre-erection camber at a degree that in some cases is worth considering. The selection of the creep and shrinkage model has a marked influence on the prediction of pre-erection camber development. The time when initial camber is measured appears to be an important parameter since marked differences were found between predicted camber at release and predicted camber during the first 10 days. The influence of the model for calculating modulus at prestress release and 28 days has a marked effect on pre-erection camber predictions, whereas the model to predict the variation of modulus with time had negligible effect. The inclusion of overstrength factors also had a marked effect on pre-erection camber predictions. The default set of models used in most predictions presented in this report is shown in Figure 3.17 except that the no overstrength factors were considered. This selection resulted in reasonable predictions of pre-erection camber as will be demonstrated later and is therefore recommended for design. The other models may be investigated to quantify the variability of pre-erection camber predictions. The proposed prediction methodology (Approach No. 6) provides the user the flexibility of accounting for the influence of all the abovementioned factors. In addition, the proposed prediction methodology provides the necessary beam camber data such that the profile of the beam can be plotted for various stages of deflection and slab and screed elevations can be determined accordingly.

CHAPTER 6: Evaluation of Factors that Affect Camber and Deflections in the Composite System

6.1. Introduction

Various factors affect displacements in a composite bridge superstructure that features prestressed concrete or steel beams and a cast in place concrete deck. Beam creep as a result of the prestressing force (for PC beams), and positive temperature gradients cause negative curvature (i.e. upward deflection). Conversely, the self-weight of deck and beam and associated creep, differential shrinkage between the deck and beam, and negative temperature gradients cause positive curvature (i.e. downward deflection). Even for PC beams, differential shrinkage between the deck and beam occurs since the majority of the shrinkage in the beam has already taken place when the deck is cast, while the shrinkage of deck begins after the moist curing of the deck ends. A similar effect applies to composite steel beams with the exception that differential shrinkage in this case is more pronounced since steel beams do not shrink.

When PC beams are considered, differential shrinkage and creep between the deck and beam strongly depend on the age of the beam at deck placement. This age is perhaps the most critical and influential factor in determining beam deflections in the composite bridge superstructure. The type of bridge superstructure also affects the magnitude of beam deflections in the composite system and must be considered. For example, beam deflections in a two span continuous bridge differ from those in a simply supported bridge. When composite steel beams are considered, the deflection components involving the prestressing effect, beam creep and beam shrinkage can be ignored, and the same methodology can be used to compute deflections at any point in time while accounting for deck shrinkage and creep. The prediction methodology presented in this report allows the calculation of deflections at any point in time and the inclusion of appropriate shrinkage and creep coefficients for each component.

6.2. Sensitivity Analysis

Overview: The influence of various factors that affect camber and displacements in the composite system was evaluated using Approach No. 6 of the proposed prediction methodology. The influence of some factors was expressed in % change in net camber as a function of a given % change in the factor under consideration. This influence was expressed in terms of the average, minimum and maximum change, standard deviation and covariance for all projects considered. The considered final service time for this evaluation was 75 years. The results of this analysis are shown in Table 6.1. It should be noted that minimum and maximum changes were sometimes driven by the net camber for the baseline case. For example, if the net camber for the baseline is close to zero, even a small change in camber or displacement would result in a high % change and can therefore be misleading. The provision of the average change and covariance addresses this to a certain degree. The induced change was $\pm 10\%$. Factors that led to a single digit average % change in net camber and displacement due to the induced 10% change include: beam overhang length at precast facility (i.e. storage conditions), transfer length, debonded length, deck modulus, and beam overhang length at the bridge site. Factors that led to a double digit average % change in net camber and displacement due to the induced 10% change include: beam concrete compressive strength at release and 28 days, beam concrete unit weight, beam concrete modulus of elasticity at release and 28 days, location of harping point, beam spacing, compressive strength of deck at 28 days, and unit weight of deck. Factors that led to a triple digit average % change in net camber and displacement due to the induced 10% change include: beam length, and prestressing force.

Additional Information: The parameter with the strongest influence was beam length. For example, a $\pm 10\%$ change in beam length resulted in a change in camber and displacements that

varied from -3155.6% to 1866.7%, suggesting that long-term camber and displacements are very sensitive to beam length. The average change in net camber or displacement at service at 75 years for a +/-10% change in beam length was -199% and 110% with -362% and 395% covariance, respectively. The beams used in project M5 dominated the lower bound for this analysis. The net predicted camber at 75 years for these beams is 0.06 in. (i.e. nearly zero). Therefore, a 1 in. change in net camber due to a change in beam length or other factors causes a $(1-0.06)/0.06 \times 100 = 1566\%$ in net camber. Therefore, when the net camber value that serves as the baseline for comparison is small, any deviation from that value results in a large % change.

A +/-10 % in the initial compressive strength led to a change in camber and displacements that varied from -455.6% to 566.7%. Although, the average change in net camber or displacement at service at 75 years for a +/-10% change in f'_{ci} was -47.1% and 57.9% with -212% and 215% covariance, respectively.

Similarly, a +/-10 % change in the 28 days design compressive strength led to a change in camber and displacement that varied from -655.6% to 566.7%. Although, the average change in net camber or displacement at service at 75 years for a +/-10% change in f'_c was 23.3% and 18.4% with 215% and 278% covariance, respectively.

The magnitude of the prestressing force also had a strong influence on long-term camber and displacements at service. For example, a +/-10 % change in $P_{jacking}$ led to -1411% to 1400% change in camber and displacements at service. Although, the average change in net camber or displacement at service at 75 years for a +/-10% change in $P_{jacking}$ was 118.1% and -119% with 264% and 265% covariance, respectively.

Beam concrete unit weight also had a strong influence on camber and displacements at service featuring a variation +/-778% due to a +/-10 % change in unit weight. The average change

in net camber or displacement at service at 75 years for a $\pm 10\%$ change in w_{beam} was $\pm 56.9\%$ with $\pm 308\%$ covariance.

The influence of the initial modulus of elasticity of beam concrete on camber and displacements at service was also significant but not as strong as that of the aforementioned parameters with the exception of concrete compressive strength. A $\pm 10\%$ change in E_{ci} led to a change in camber and displacements at service that varied from -667% to 811% . The average change in net camber or displacement at service at 75 years for a $\pm 10\%$ change in $E_{ci_{beam}}$ was $+62.6\%$ and -76.5% with 235% and -234% covariance, respectively.

Beam concrete modulus at 28 days also had a strong influence but not as strong as that of the initial modulus. A $\pm 10\%$ change in $E_{c_{beam}}$ led to a change in camber and displacements at service that varied from -656% to 566.7% . The average change in net camber or displacement at service at 75 years for a $\pm 10\%$ change in $E_{c_{beam}}$ was 45.8% and -53.2% with 277% and -276% covariance, respectively.

Beam overhang length during storage at the precast facility as well as beam overhang past the bearing points at the bridge site had a lower influence than that of all parameters considered so far. A $\pm 10\%$ change in the beam overhang length at precast facility and bridge site led to a $\pm 22\%$ and $\pm 11\%$ change in beam camber and displacements at service, respectively.

The influence of transfer length and strand debonded length on beam camber and displacements at service was also lower than that of all aforementioned parameters, with a $\pm 10\%$ change in transfer length and strand debonded length leading to a $\pm 3-7\%$ change in beam camber and displacements at service.

A $\pm 10\%$ change in the location of harping points led to -178% to 156% change in beam camber and displacements at service.

Finally, a $\pm 10\%$ change in beam spacing led to -333% to 356% change in beam camber and displacements at service concluding the sensitivity analysis conducted for concrete beam related parameters on beam camber and displacements. The sensitivity analysis continued by investigating the influence of several deck related parameters on beam camber and displacements at service.

A $\pm 10\%$ change in the 28 day deck concrete compressive strength led to -167% to 156% change in beam camber and displacements at service. Naturally, the influence of deck concrete compressive strength on beam camber and displacements is not as strong as that of the beam concrete compressive strength since the beam comprises the majority of the composite section.

Similarly, a $\pm 10\%$ change in the 28 day deck concrete modulus led to a $\pm 33\%$ change in beam camber and displacements at service. The influence of deck concrete modulus on beam camber and displacements is also not as strong as that of the beam concrete modulus since the beam comprises the majority of the composite section.

Table 6.1. Influence of various parameters on camber and displacements at service

Parameter varied	Service Camber Sensitivity Analysis				
	% Change in net camber/displacement				
	Lower bound % change for net camber	Upper bound % change for net camber	Average	St. Dev.	COV (%)
Baseline (Approach No. 6)	-	-	-	-	-
$L_G +10\%$	-3156	141	-198.7	718.8	-362
$L_G -10\%$	-258	1867	109.7	433.2	395
$f'_{ci\,beam} +10\%$	-456	-14	-47.1	99.9	-212
$f'_{ci\,beam} -10\%$	18	567	57.9	124.4	215
$f'_{c\,beam} +10\%$	3	289	23.3	64.8	278
$f'_{c\,beam} -10\%$	-38	522	18.4	122.3	666
$P_{jacking} +10\%$	22	1400	118.1	312.4	264
$P_{jacking} -10\%$	-1411	-22	-119.0	314.9	-265
$w_{beam} +10\%$	-778	-3	-56.9	175.4	-308
$w_{beam} -10\%$	4	778	56.9	175.4	308
$E_{ci\,beam} +10\%$	-667	-16	-62.6	147.4	-235
$E_{ci\,beam} -10\%$	20	811	76.5	179.3	234
$E_{c\,beam} +10\%$	7	567	45.8	127.1	277
$E_{c\,beam} -10\%$	-656	-6	-53.2	146.9	-276
$L_{overhang\,facility} +10\%$	0	22	1.7	5.2	309
$L_{overhang\,facility} -10\%$	-22	0	-1.6	5.2	-323
$L_{transfer} +10\%$	-1	0	-0.3	0.4	-140
$L_{transfer} -10\%$	0	3	0.3	0.7	219
$L_{debonded} +10\%$	-7	0	-1.3	2.0	-150
$L_{debonded} -10\%$	0	7	1.4	2.1	151
$L_{harping} +10\%$	-178	-2	-22.7	54.5	-240
$L_{harping} -10\%$	2	156	19.9	47.7	239
$s_D +10\%$	-333	-4	-30.0	74.4	-248
$s_D -10\%$	6	356	32.3	79.3	246
$f'_{c\,deck} +10\%$	3	156	14.9	34.6	232
$f'_{c\,deck} -10\%$	-167	-2	-16.0	37.1	-232
$w_{deck} +10\%$	-233	-3	-27.5	54.9	-200
$w_{deck} -10\%$	-100	233	17.2	59.2	345
$E_{c\,deck} +10\%$	0	33	4.2	8.1	192
$E_{c\,deck} -10\%$	-33	0	-4.0	7.6	-188
$L_{overhang\,bridge} +10\%$	0	11	1.5	2.7	183
$L_{overhang\,bridge} -10\%$	-11	3	-1.0	2.6	-264

L_G = Beam length, $f'_{ci\,beam}$ = compressive strength of beam concrete at release, $P_{jacking}$ = prestressing force at jacking, w_{beam} = unit-weight of girder, $E_{ci\,beam}$ = modulus of elasticity of girder concrete at release, $E_{c\,beam}$ = modulus of elasticity of girder concrete at 28 days, $L_{overhang\,facility}$ = beam overhang length at precast facility, $L_{overhang\,bridge}$ = beam overhang length at bridge site, $L_{transfer}$ = transfer length, $L_{debonded}$ = debonded length, $L_{harping}$ = location of harping point measured from girder end, s_D = beam spacing, $f'_{c\,deck}$ = deck concrete compressive strength at 28 days, w_{deck} = unit-weight of deck concrete, $E_{c\,deck}$ = modulus of elasticity of deck concrete at 28 days.

6.2.1. Influence of Creep and Shrinkage Model on Service Camber

The influence of creep and shrinkage models for beam and deck concrete on camber and displacements at service was investigated by predicting the full displacement history for the prestressed concrete girders used in the S11 project from the time they were fabricated to the assumed end of service life for this bridge (75 years). This comparison is shown in Figure 6.1a. Predictions based on Bazant and Baweja B3 model and those based on GL 2000 model provided much higher estimates of pre-erection camber than measured values and were therefore removed from this comparison. Figure 6.1a suggests that the selection of creep and shrinkage models has a marked influence on the full displacement history of the prestressed concrete beams. It is worth noting how the AASHTO LRFD 2020 model based on specifications provided in the body of AASHTO LRFD 2020 and fib MC 2010 model predict an increase in downward displacements after placement of deck whereas the rest of the models, including the one based on specifications provided in the commentary of AASHTO LRFD 2020 predict an increase in camber. Similarly, the predicted camber after 75 years is generally similar between the fib MC 2010 model, AASHTO LRFD 2020 commentary and CEB MC90-99 model. Although, this value is much different than that predicted by the AASHTO LRFD 2020 body model. The CEB MC90 and ACI 209R-92 models also provide similar predictions of camber after 75 years, although different from the rest of the models. Finally, all models show that there is generally some stability in net camber after one year.

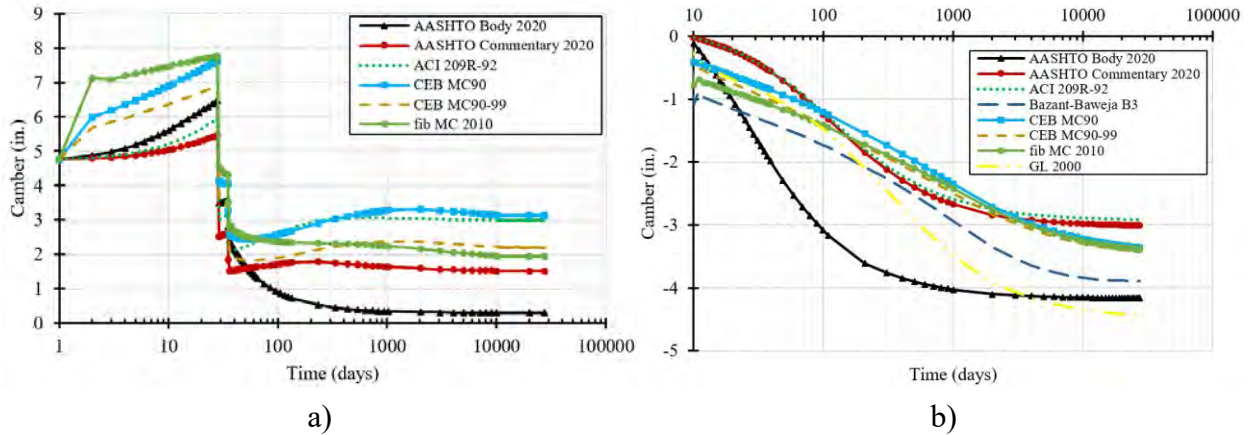


Figure 6.1. Influence of creep and shrinkage problems on the full displacement history of: a) prestressed concrete beams used in the S11 project, and b) steel beams used in the M20 project

A similar comparison was conducted for the steel beams used in the M20 project (Figure 6.1b). In this comparison it was assumed that the net camber after the placement of deck and superimposed dead loads was zero. This was done due to the fact that there was some ambiguity as to what the pre-erection camber in the steel beams was and whether this pre-erection camber included the effects of beam self-weight. Therefore, the displacement history starts after the placements of superimposed dead loads (barrier and overlay) (i.e. after the opening of the bridge to service). Predictions based on Bazant and Baweja B3 model and those based on GL 2000 model are included in this comparison. As can be seen, the selection of creep and shrinkage model influences the entire displacement history leading to different predictions. The cause of these differences is primarily due to differences in the predicted shrinkage in the deck, which leads to differential shrinkage between the concrete deck and the steel beam. Differential shrinkage induced creep is a second reason for the differences in predicted displacements, although its influence is not as strong as that of deck shrinkage.

6.2.2. Influence of Model to Predict Modulus of Elasticity at Prestress Release and 28 Days

The influence of the model to predict modulus of elasticity at prestress release and 28 days on camber history was evaluated by running the proposed prediction methodology (Approach No.

6) four times for the S11 project; one time with the benchmark model (AASHTO LRFD 2020) and three other times using the ACI 318-19 model, ACI 363R-92 model, and fib MC 2010 model, respectively. When using the AASHTO LRFD 2020 model, the aggregate factor, K_1 , was taken equal to 1.0 and concrete unit weight, w_c , was calculated using the compressive strength dependent function as provided in Eqns. 3-104 and 3-105. Additionally, the ACI 209R-92 model with back-calculated parameters was selected to capture the variation of modulus with time, no overstrength factors were used, and the body provision of AASHTO LRFD 2020 model for creep and shrinkage were used as part of the benchmark model. When using the fib MC 2010 model, the aggregate type, α_E , was assumed to be quartzite. The number of beam sections along half the span was selected as 51 and the proposed time step generation method was used. Figure 6.2a shows the comparison between moduli of elasticity at prestress release and 28 days using various models. Figure 6.2b shows the difference in camber magnitude when each model is considered. Figure 6.2b suggests that the selection of the model for modulus of elasticity at prestress release and 28 days has a noteworthy influence on camber history with the ACI 318-19 model resulting in differences up to 148% and ACI 363R-92 resulting in differences up to 74%. In addition, it is interesting to note that although there is direct correlation between the predicted camber and modulus of elasticity up to pre-erection such that the lower the modulus yields the highest pre-erection camber, this trend is not observed in service camber. This can be explained by the fact that depending on which prediction model is employed for modulus of elasticity, the force history for the girder due to application of deck, barrier and overlay loads is affected differently, and therefore, it leads different camber and displacement results for the service state.

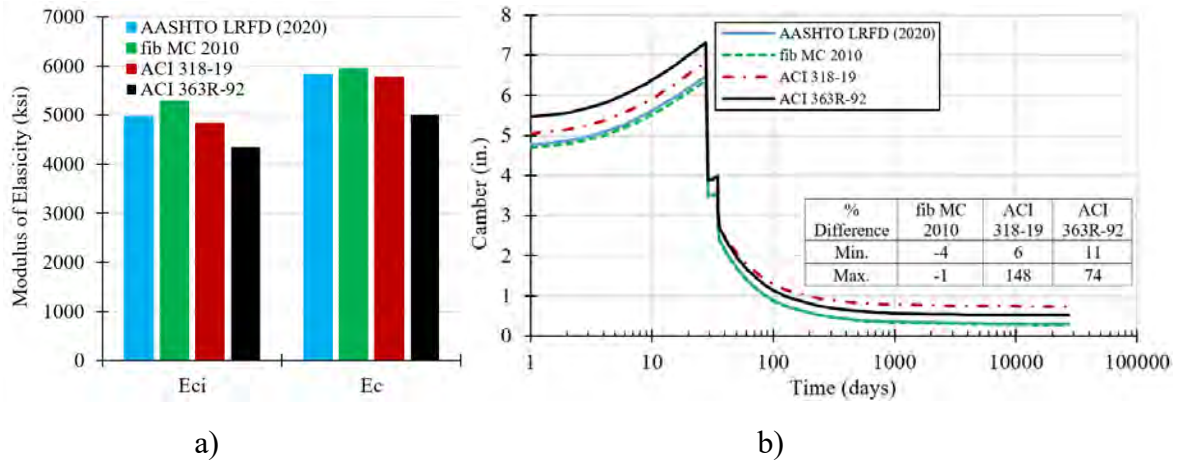


Figure 6.2. a) Comparison of calculated modulus of elasticity at prestress release and 28 days based on different models, and b) influence of model to predict modulus of elasticity at prestress release and 28 Days on camber history for S11 project

6.2.3. Influence of Model for Capturing the Variation of Modulus with Time

The influence of the model to capture the variation of modulus with time on camber history was investigated by running the algorithm twice: once based on the benchmark model (i.e. ACI 209R-92), and another time using the fib MC 2010 model. When using the benchmark model, the AASHTO LRFD 2020 model for creep and shrinkage as well as prediction of modulus at prestress release and 28 days were used. The number of beam section along half the span was selected as 51 and the proposed time step generation method was used. The results are shown in Figure 6.3, which shows that predictions based on both models are almost identical, therefore the selection of one model or the other is inconsequential. The % difference in camber history predictions is less than 0.5%.

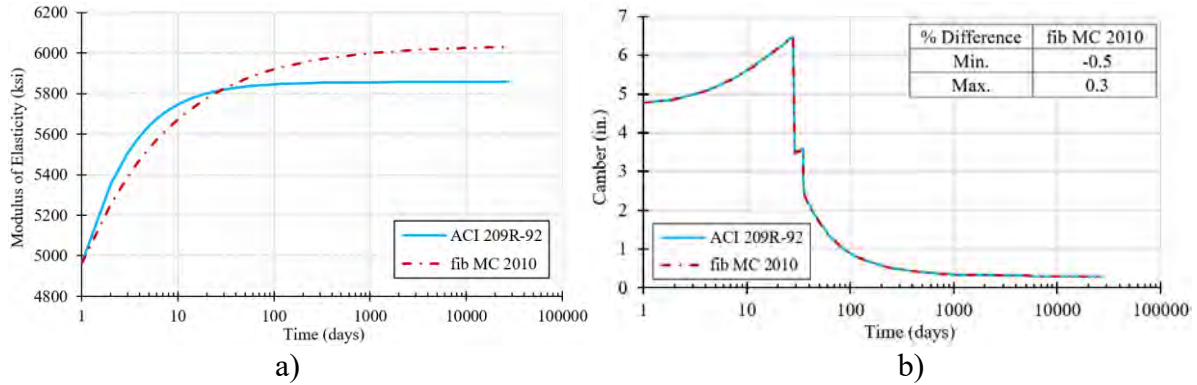


Figure 6.3. a) Comparison of predictions for development of modulus of elasticity with respect to time based on different models, and b) influence of time dependent modulus model on camber for S11 project

6.2.4. Influence of Overstrength Factors

The influence of overstrength factors for beam concrete compressive strength at prestress release and 28 days was investigated by running the algorithm once without overstrength factors and another time with Michigan based overstrength factors (Eqns. 4-1 and 4-2). It should be noted that the use of overstrength factors for f'_{ci} affects the calculation of modulus at prestress release, E_{ci} , which affects the calculation camber history. Additionally, the use of overstrength factors affects both f'_{ci} and f'_c , which in turn affect moduli of elasticity at prestress release and 28 days. These moduli of elasticity at these two times serve as anchor points for the model that predicts the development of modulus with time. The benchmark model was based on no overstrength factors, AASHTO LRFD 2020 model for modulus at release and 28 days as well as creep and shrinkage, and the ACI 209R-92 model for capturing the variation of modulus with time. The only change in the second analysis was the inclusion of the overstrength factors for f'_{ci} and f'_c . The number of beam sections along half the span was selected as 51 and the proposed time step generation method was used. The results are shown in Figure 6.4. The inclusion of overstrength factors does have a marked effect on camber history predictions with differences between the two models being up to 160%. As expected, the inclusion of overstrength factors results in lower camber predictions.

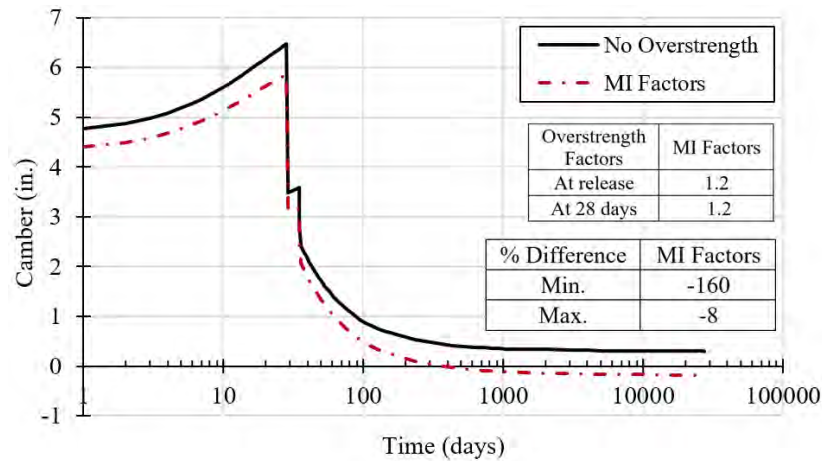


Figure 6.4. Influence of inclusion of overstrength factors for S11 project

6.2.5. Influence of the Time when the Deck is Placed

The influence of the time when the deck is placed on beam camber and displacements was investigated by varying the time when the deck was placed from 20 days to 55 days and to 90 days. This exercise was conducted for two projects, S10-2, and S11. Figure 6.5a and Figure 6.5b shows the full displacement history for the three considered deck placement times for S10-2 and S11 projects, respectively. As can be seen, the longer the time when the deck is placed, the higher the pre-erection camber. While this is intuitive, interestingly, a delay in the time when the deck is placed causes the final camber to be lower. This seemingly counterintuitive finding is due to the fact that differential shrinkage between the deck and the girder dominates the prestressing force induced camber growth at a greater extent when the placement of the deck is delayed. It is also interesting to note how camber and displacements after 75 years appear to be rather similar regardless of when the deck is placed.

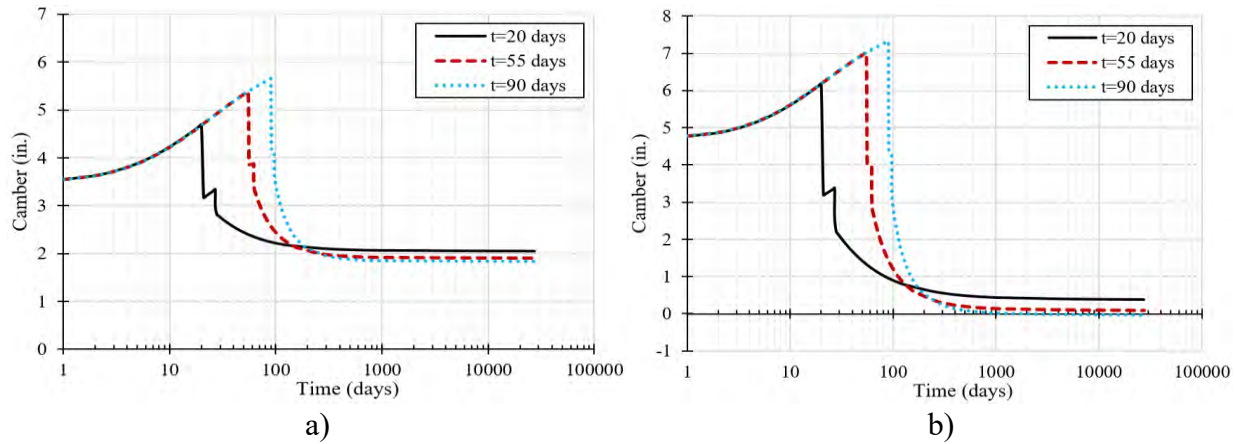


Figure 6.5. Influence of the time when the deck is placed on beam camber and displacements at service for: a) S10-2 project; b) S11 project

6.2.6. Influence of the Temperature Gradient

The influence of temperature gradients on beam camber and displacements was investigated by considering the S10-2, S11, and M20 projects and by using Approach No. 6 of the proposed prediction methodology. Camber and displacements were first calculated at various points in time by ignoring the influence of temperature gradients to establish a baseline, and then were recalculated by considering both positive and negative temperature gradients based on solar radiation zone III described in AASHTO LRFD Bridge Design Specifications (2020) section 3.12.3. The results of this analysis are shown in Figure 6.6. The evolution of camber in both S10-2 and S11 projects is characterized by the traditional growth between the detensioning of the strands and the placement of the deck, followed by a sudden decrease in camber due to the placement of the deck, and a further decrease due to time dependent effects. The influence of both negative and positive temperature gradients is rather consistent throughout the life of the beams with the positive temperature gradients having a stronger influence on camber and displacements. Therefore, if construction activities are conducted during the summer months, the contractor should be mindful of the impact that temperature gradients can have on pre-erection camber and orchestrate activities accordingly. Figure 6.6c suggests that the influence of temperature gradients

on the M20 project, which features steel beams was similar to that observed in the prestressed concrete beam projects (S10-2 and S11). The camber and displacement history for the steel beams in the M20 project start at deck placement since it is assumed that the beams come with a certain fabricated pre-erection camber. For S10-2, S11, and M20 projects, the absolute maximum % difference values in camber caused by positive temperature gradients are 20%, 217%, and 600% and those caused by negative temperature gradients are 6%, 65%, and 180% respectively.

The influence of temperature gradient appears to be large in terms of % change with the baseline value. However, this is because in some cases the deflection of the beams is either small or switches from an upward displacement (camber) to a downward displacement. This results in the baseline value used for comparison to be small and yields large % differences in camber predictions when in fact the difference in camber or displacement magnitudes is smaller. The proposed prediction methodology has the ability to quantify camber and displacements due to temperature gradients for each zone described in AASHTO LRFD Bridge Design Specifications (2020) section 3.12.3, and this information can be used to plan for certain construction operations such as the placement of the deck.

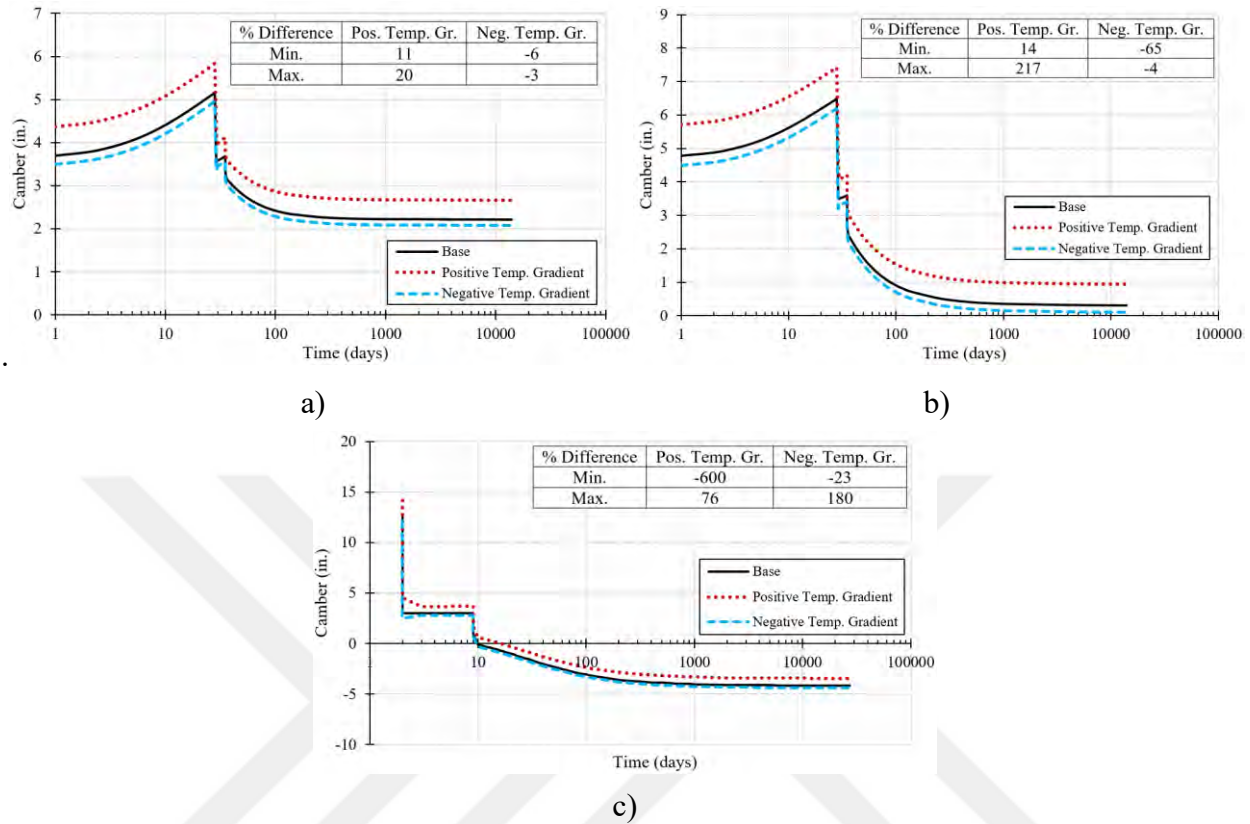


Figure 6.6. Influence of temperature gradients on beam camber and displacements at service: a) S10-2 project; b) S11 project, c) M20 project

6.3. Summary and Conclusions

The influence of various factors that affect camber and displacements in the composite system was evaluated using Approach No. 6 of the proposed prediction methodology. The influence of some factors was expressed in % change in net camber as a function of a given % change in the factor under consideration. This influence was expressed in terms of the average, standard deviation, covariance, minimum, and maximum change for all projects considered. It should be noted that minimum and maximum changes were sometimes driven by the net camber for the baseline case. For example, if the net camber for the baseline is close to zero, even a small change in camber or displacement would result in a high % change and can therefore be misleading. The provision of the average change addresses this to a certain degree. The induced change was $\pm 10\%$.

Factors that led to a single digit average % change in net camber and displacement due to the induced 10% change include: beam overhang length at precast facility (i.e. storage conditions), transfer length, debonded length, deck modulus, and beam overhang length at the bridge site.

Factors that led to a double-digit average % change in net camber and displacement due to the induced 10% change include: beam concrete compressive strength at release and 28 days, beam concrete unit weight, beam concrete modulus of elasticity at release and 28 days, location of harping point, beam spacing, compressive strength of deck at 28 days, and unit weight of deck.

Factors that led to a triple digit average % change in net camber and displacement due to the induced 10% change include: beam length, and prestressing force.

The influence of the selected creep and shrinkage model on the full displacement history of a prestressed concrete beam bridge was investigated and it was concluded that this selection has a marked influence on the beam displacement history. Some models result in rather similar displacements after 75 years despite initial differences in pre-erection camber and net displacements after deck placement.

The selection of the model to predict modulus of elasticity at prestress release and 28 days had a marked effect on camber history with differences from the baseline being as high as 148%. The influence of the model to predict the variation of modulus with time was inconsequential.

The inclusion of overstrength factors also had a marked effect on camber history with differences with the case when they are not included being as high as 160%.

Influence of deck placement time on the full beam displacement history was investigated and it was concluded that while pre-erection camber is highly influenced by it, camber and displacements after 75 years were rather similar. The influence of temperature gradients was rather uniform throughout the displacement history of the beam with positive temperature gradients

having a higher influence on camber and displacements compared to negative temperature gradients.



CHAPTER 7: Evaluation of Factors that Affect Beam Rebound and Behavior after Deck Replacement

7.1. Introduction

Intuitively, factors affecting existing steel and concrete beam rebound, net camber after deck removal, and net camber before new deck placement include: the weight of the existing deck, the age of beam concrete when the deck is removed, the time between the removal of exiting deck and the placement of the new deck, conditions of the beam and deck concrete at the time of deck removal, including any changes in geometry or material properties such as those caused by deterioration, and the type of structure (simply supported or continuous).

The proposed prediction methodology provides the means to estimate beam rebound, net camber after deck removal, and net camber before new deck placement for simply supported bridges that feature prestressed concrete and steel beams. The estimation was done using Approach No. 5 and Approach No. 6 to see the difference in results. In the majority of the analysis conducted, it was assumed that the old deck would be replaced after 50 years and that the time between the old deck removal and new deck placement is 10 days. The influence of deck replacement time was investigated by varying it from 40 years to 60 years in increments of 5 years. It was further assumed that for the first 7 days after the placement of the new deck, the new deck will be moist cured such that no deck shrinkage takes place. The bridge structures were assumed to be simply supported and that no significant beam deterioration has taken place in terms of section loss and material degradation. The only consequential deterioration is assumed to have taken place in the deck thus prompting a deck replacement.

7.2. Comparison of Rebound and Net Camber after Deck Removal

Table 7.1 provides a comparison between beam rebound, net camber after deck removal, and net camber before new deck placement. This exercise was conducted for four projects using

Approach No. 5 and Approach No. 6 of the proposed prediction methodology. There are some differences between the predicted elastic change in camber (rebound) when Approach No. 5 and Approach No. 6 are used. Approach No. 6 leads to smaller rebounds compared to Approach No. 5. This difference is due the different effective moduli of elasticity used in Approach No. 5 and Approach No. 6, respectively. Recall that the effective modulus is a function of the creep coefficient, and Approach No. 5 and Approach No. 6 use different creep coefficients because the former is based on a single creep curve whereas the latter is based on multiple creep curves. Another reason why the elastic change in displacement (rebound) is different in Approach No. 5 and Approach No. 6 is due to the fact that the internal axial forces and bending moments in the deck, which are considered as newly applied forces on the non-composite (i.e. beam and strand) beam section, are different. Since they are different, the rebound caused by their removal will be different.

In the light of this discussion, it is natural to expect differences in the net camber after deck removal since the history of forces in the beam and deck is different. Additionally, predictions based on Approach No. 5 suggest that there is no change between net camber after deck removal, and net camber before new deck placement whereas Approach No. 6 suggests that there is a slight camber growth. This comparison highlights the fundamental differences between the approaches used in Approach No. 5 and Approach No. 6. The reason why the net camber after deck removal and net camber before new deck placement is identical in Approach No. 5 is due to the fact that the remaining creep coefficient calculated based on a single creep curve for beam concrete is nearly zero. Approach No. 6 on the other hand uses a different creep curve for the unloading of the beam due to the removal of the deck, and this different creep curve suggests that concrete can still creep after 50 years.

Table 7.1. Comparison of rebound, net camber after deck removal, and net camber before new deck placement

Project Name	Approach No. 5			Approach No. 6		
	Elastic change (rebound) in camber* (in.)	Net camber after deck removal ⁺ (in.)	Net camber before new deck placement ⁺ (in.)	Elastic change (rebound) in camber* (in.)	Net camber after deck removal ⁺ (in.)	Net camber before new deck placement ⁺ (in.)
M5	3.67	2.80	2.80	3.39	1.51	1.81
TSO	1.52	2.33	2.33	1.43	2.00	2.13
S10-2	2.60	4.84	4.84	2.43	4.44	4.64
S11	4.65	5.53	5.53	4.44	4.74	5.10

*Old deck is assumed to be removed at 50 years.

⁺Elapsed time between after deck removal and before new deck placement is assumed to be 10 days.

Net camber prior to placement of the first deck was compared to the net camber after the removal of the deck to determine whether the beam would rebound to its original position. Five prestressed concrete beam and one steel beam project were considered. The results are shown in Table 7.2. The penultimate column shows the ratio (f/a) of net camber after removal of deck (and consequently locked in forces in the deck) to the net camber prior to the placement of the first deck. For the steel beam project this ratio is equal to 1.00. This is logical because steel beams do not creep or shrink, and after the concrete deck is removed, there are no obstacles for the beam to rebound to its original position. In reality, this may not be the case due to a change in the boundary conditions (i.e. pin supports may not behave as pin supports), and possible plate action due to the presence of diaphragms which may restrain the movement of some beams relative to adjacent beams. For the prestressed concrete beams, the ratio f/a varies from 0.47 to 0.92 (i.e. <1.0), which is an indication that after the deck is placed, the beam continues to deflect downwards due to differential shrinkage, differential shrinkage induced creep as well as girder self-weight induced creep. This additional downward deflection, combined with the smaller immediate rebound when the deck is removed due to the increased modulus of the beam, results in f/a ratios that are lower than 1.0.

Table 7.2. Comparison of net camber prior to placement of deck to net camber after removal of deck

Project Name	Placement of Deck and Barrier (in.)			Removal of Barrier and Deck (in.)			Ratio = Removal/Placement	
	Prior to Placement of Deck (a)	After Placement of Deck (b)	After Placement of Barrier (c)	Prior to Removal of Barrier (d)	After Removal of Barrier (e)	After Removal of Deck and Locked in Forces (f)	f/a	e/b
S11	6.47	3.53	3.59	0.93	1.67	4.99	0.77	0.47
S10-2	4.93	3.42	3.17	2.30	2.62	4.56	0.92	0.77
M5	3.69	2.21	1.15	-1.31	-0.37	1.75	0.47	-0.17
TSO	2.47	1.60	1.45	0.75	0.95	2.08	0.84	0.59
S05	5.74	1.99	1.03	-1.58	-0.69	3.35	0.58	-0.35
M20*	11.68	2.07	0.00 ⁺	-3.86	-2.97	11.68	1.00	-1.44
*Steel project; ⁺ Fabricated camber is arranged such a way that after barrier load is applied, net camber is zero.								

7.3. Influence of Deck Replacement Time

The influence of the time when the old deck is replaced on the magnitude of rebound, net camber after deck removal, and net camber before new deck placement was investigated using Approach No. 6 by considering various deck replacement times (40-60 years in increments of 5 years). This exercise was conducted for the S11 project. As can be seen from Table 7.3, deck replacement time had no influence on the magnitude of beam rebound and net camber after deck removal provided that the deck is replaced at least after 40 years. Again, this is due to the fact that the modulus of the beam has reached a stable value after 40 years. This is evident from all full beam displacement history curves shown in previous figures. Deck replacement time had a minor influence on the net camber before new deck placement with greater deck replacement times resulting in slightly lower net cambers before new deck placement. However, as shown in Table 7.3, this influence is minimal. This minimal reduction in net camber before new deck placement as the deck replacement time increases is due to the fact that concrete creeps less when the load is applied or removed at a later time rather than at an earlier time.

Table 7.3. Influence of deck replacement time on beam rebound, net camber after deck removal, and net camber before new deck placement based on Approach No. 6 (S11 project)

Time at old deck is replaced (years)	Elastic change (rebound) in camber* (in.)	Net camber after deck removal ⁺ (in.)	Net camber before new deck placement ⁺ (in.)
40	4.44	4.74	5.11
45	4.44	4.74	5.11
50	4.44	4.74	5.10
55	4.44	4.74	5.10
60	4.44	4.74	5.09

*Old deck is assumed to be removed at 50 years.

⁺Elapsed time between after deck removal and before new deck placement is assumed to be 10 days.

7.4. Summary and Conclusions

The influence of solution method (Approach No. 5 vs. Approach No. 6) on beam rebound, net camber after deck removal, and net camber before new deck placement was investigated. It was determined that Approach No. 6 leads to smaller rebounds compared to Approach No. 5. Additionally, predictions based on Approach No. 5 showed that there is no change between net camber after deck removal, and net camber before new deck placement whereas Approach No. 6 suggests that there is a slight camber growth.

Steel beams were able to rebound to their original position once the deck was removed whereas prestressed concrete beams rebounded to 47-92% of their original position depending on whether differential shrinkage or prestressing induced creep dominated the behavior of the composite system.

When Approach No. 6 was used, deck replacement time had no influence on the magnitude of beam rebound and net camber after deck removal provided that the deck is replaced at least after 40 years. Deck replacement time had a minor influence on the net camber before new deck placement with greater deck replacement times resulting in slightly lower net cambers before new deck placement.

CHAPTER 8: Evaluation of Various Prediction Methodologies for Time-Dependent Flexural Deformation

8.1. Introduction

The purpose of this chapter is to:

1. compare the measured camber at release and pre-erection camber provided as part of the dataset with predictions based on PCI multiplier Method, MDOT multiplier method, and the proposed methodology,
2. compare the measured camber histories collected from precast facilities and obtained from literature with predictions based on proposed prediction methodology (Approach No. 6) using various creep and shrinkage models,
3. compare the measured camber histories collected from precast facilities and obtained from literature with predictions based on various approaches within the proposed prediction methodology (Approaches No. 1 to No. 6),
4. compare camber predictions based on various approaches within the proposed prediction methodology to determine the impact of various assumption,
5. compare predictions of prestress losses based on AASHTO LRFD Bridge Design Specifications (2020) and those obtained using the proposed prediction methodology,
6. compare predictions of concrete stresses based on various approaches within the proposed prediction methodology to determine the impact of various assumption,
7. evaluate the impact of method to generate time steps using proposed prediction methodology (Approach No. 6),
8. evaluate the impact of number of beam sections considered in the analysis using proposed prediction methodology (Approach No. 6), and

9. present the Time-dependent Multiplier Method and evaluate the results based on provided dataset (Table 4.1).

8.2. Evaluation of Prediction for Camber at Release

Measured camber at release data provided by the fabricators was compared with predicted camber at release using Approach No. 6. The predicted camber at release was calculated once based on measured beam properties and another time based on specified properties. The measured properties in question are the concrete compressive strength at release, length of beam, concrete unit weight, and magnitude of prestressing force. In all predictions, the modulus of elasticity of concrete at release was determined using AASHTO's (2020) formula (3-104 and 3-105), which is a function of concrete compressive strength, unit weight, and aggregate factor. The measured camber was compared to predicted camber using Approach No. 6. When conducting this comparison, it was assumed that camber at release was measured as soon as the strands were detensioned, thus allowing no time for the concrete to creep and shrink. Measured camber data shows that this was not always the case because the fabricator has up to seven days to report camber at release according to MDOT's Special Provisions for Structure Survey during Construction. Moreover, it was assumed that the camber measurements were taken with respect to girder ends, not temporary support locations. In addition to MDOT database, the first measurements of the periodic camber measurements (Hinkle (2006) and TSO project) were considered as camber at release by considering the time-dependents effects up to the day in which the camber measurements were taken. The measured camber at release was also compared with the anticipated camber specified in the design drawings, if available.

As stated earlier, for the prestressed concrete beams fabricated for Michigan projects, information on beam storage and support conditions when camber at release was measured was

not provided as part of the camber data set. However, in several cases bridge drawings specified where such supports should be placed. Therefore, predictions were based on the assumption that such specifications were followed during beam fabrication and considered the placement of beam supports after fabrication. When camber at release was measured for the beams fabricated by PSI Windsor, the beams had been removed from the prestressing bed and stored on wooden blocks positioned 3 feet away from the ends of the beam. On the other hand, database for the beams obtained from Hinkle (2006) did not include the information about the location of temporary supports, thus it was assumed that these beams were supported at girder far ends. This support condition was taken into consideration when estimating displacements due to self-weight for this project. Furthermore, 51 sections were considered along the beam half-span in the analysis.

Summary of camber at release predictions

The comparison between measured, predicted (based on proposed methodology, Approach No. 6), and design (specified in drawings) camber at prestress release was conducted. The results of this comparison are provided in Figure 8.1 and Table 8.1. Figure 8.1 provides a graphical comparison between measured and predicted camber at release for each beam using measured and specified properties. Design camber values specified in drawings are also included. Table 8.1 provides a summary of statistical indicators for the beam-by-beam evaluation. The average ratio of measured over predicted camber when the proposed method was used is 1.07 and 0.98 when measured and specified properties were employed with 22% and 23% COVs, respectively. These statistics suggest that using specified properties give better results when compared to using measured properties, however results obtained using specified properties yield less consistent results when compared to results obtained using measured properties. The ratio of measured over predicted camber using the current MDOT method (i.e. design camber) is 1.13 and the COV is

42%. This suggests that the current MDOT method is less accurate and less consistent than the proposed methodology regardless of whether specified or measured properties are used. The range of measured over predicted camber when the proposed methodology is used is 0.42-1.57 when measured properties are employed, and 0.41-1.50 when specified properties are utilized. When MDOT's method is used the range of measured over predicted camber is 0.48-3.36. These statistics suggest that the proposed methodology contains the error in prediction much better than the current MDOT method.

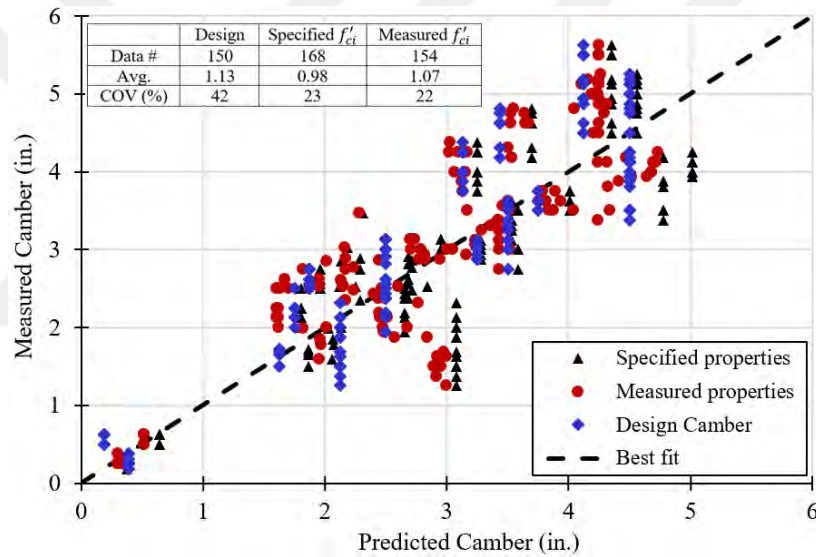


Figure 8.1. Comparison of measured, predicted, and design camber at prestress release

Table 8.1. Summary of camber at prestress release predictions

Statistical Indicators	Camber at Release Ratio		
	Δ_m/Δ_d	Δ_m/Δ_{p-m}	Δ_m/Δ_{p-s}
Min.	0.48	0.42	0.41
Max.	3.36	1.57	1.50
Avg.	1.13	1.07	0.98
St. Dev.	0.47	0.24	0.23
COV (%)	42	22	23
# of beams	150	154	168

Δ_m = measured camber (in.); Δ_d = design camber (in.); Δ_{p-m} = predicted camber based on measured properties (in.); Δ_{p-s} = predicted camber based on specified properties (in.)

8.3. Evaluation of Predictions for Pre-erection Camber

Measured pre-erection camber was compared with predicted pre-erection camber using the proposed methodology, the PCI multiplier method, and the MDOT multiplier method to demonstrate the higher accuracy of the proposed methodology. When the PCI and MDOT multiplier methods were used, camber at release was based on predictions obtained using the Approach No. 6 to contain the source of error solely to that introduced by the multiplier as opposed to compounding the errors identified in camber at prestress release predictions. Predictions based on the proposed methodology were once based on the specified properties and another time on the measured properties. Creep coefficients, shrinkage strains, and modulus of elasticity were based on the models provided in AASHTO LRFD Bridge Design Specifications (2020). Variation of modulus of elasticity with time was based on ACI 209R-92 model, and the coefficients α and β were back-calculated using the modulus of elasticity at prestress release and 28 days as anchor points. In the analysis, 51 sections were considered along the beam half-span. Assumptions related to beam storage conditions stated in evaluation camber at release results in section 8.2, are valid in this evaluation as well. Moreover, for the dataset which contains periodic camber measurements (TSO project, Hinkle (2006) and Saginaw Bridge project), each measurement was considered as a separate data point when pre-erection camber evaluation was conducted.

Summary of pre-erection camber predictions

Figure 8.2 and Table 8.2 provide a summary of the pre-erection camber predictions. The average ratio of measured over predicted camber when measured and specified properties are used is 1.06 and 0.97 and the corresponding COVs are 17% and 18%, respectively. The average ratio of measured over predicted camber when the PCI multiplier method is used based on measured and specified properties is 0.82 and 0.77, respectively, and the corresponding COVs are

18% and 18%, respectively. The average ratio of measured over predicted camber when the MDOT multiplier method is used based on measured and specified properties is used is 0.68 and 0.67, respectively, and the corresponding COVs are 20% and 18%, respectively. These statistics suggest that the proposed prediction methodology results in more accurate and consistent predictions of pre-erection camber compared to the PCI multiplier method and MDOT multiplier method regardless of whether measured or specified properties are used. In addition, using specified properties give better results when compared to using measured properties, however results obtained using specified properties yield less consistent results when compared to results obtained using measured properties.

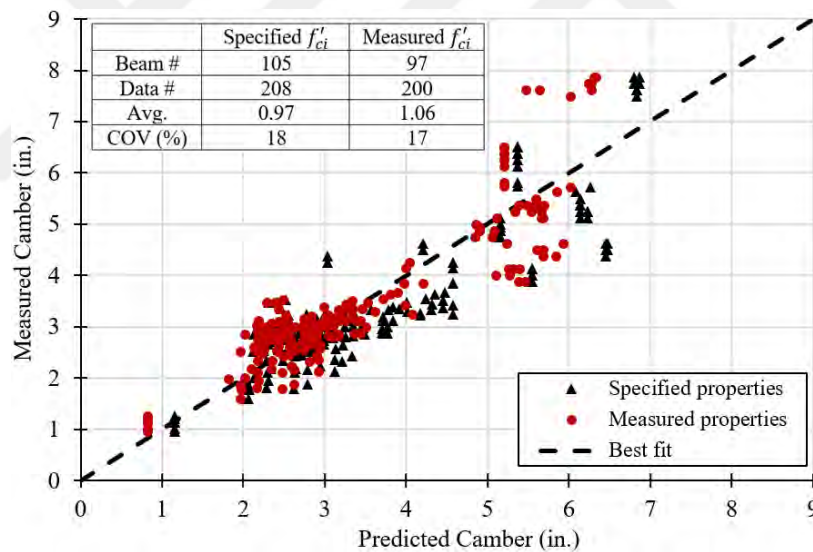


Figure 8.2. Comparison of measured and predicted pre-erection camber

Table 8.2. Summary of pre-erection camber predictions

Statistical Indicators	Pre-erection Camber Ratio					
	Δ_m/Δ_{p-m}	Δ_m/Δ_{p-s}	Δ_m/Δ_{PCI-m}	Δ_m/Δ_{PCI-s}	Δ_m/Δ_{MDOT-m}	Δ_m/Δ_{MDOT-s}
Min.	0.71	0.67	0.48	0.46	0.38	0.40
Max.	1.53	1.50	1.36	1.13	1.30	1.00
Avg.	1.06	0.97	0.82	0.77	0.68	0.67
St. Dev.	0.17	0.18	0.14	0.14	0.14	0.12
COV (%)	17	18	18	18	20	18
# of beams	200	208	200	208	200	208

Δ_m = measured camber (in.); Δ_{p-m} = predicted camber based on measured properties (in.); Δ_{p-s} = predicted camber based on specified properties (in.); Δ_{PCI-m} = predicted camber using PCI multipliers and measured properties (in.); Δ_{PCI-s} = predicted camber using PCI multipliers and specified properties (in.); Δ_{MDOT-m} = predicted camber using MDOT multipliers and measured properties (in.); Δ_{MDOT-s} = predicted camber using MDOT multipliers and specified properties (in.)

8.4. Evaluation of Predicted Camber Growth Curve

Camber growth curves using proposed prediction methodology (Approach No. 6) together with the various creep and shrinkage models were developed and compared with the periodic camber measurements. The measurements in question were either collected by the research team (Tiffin Street Overpass project) or provided by the researchers (Branson et al. 1970, Tadros et al. 1977, Bradford and Gilbert 1991, Hinkle 2006 and Fan et al. 2010). The data for the Tiffin Street Overpass project and that was provided by Hinkle (2006) includes camber measurements up to pre-erection meaning that no camber measurements after deck is placed are made available. On the other hand, the data provided by Branson et al. (1970), Tadros et al. (1977), Bradford and Gilbert (1991), and Fan et al. (2010) includes camber measurements for composite (i.e. beam and deck) structure. During the analysis, the modulus of elasticity of girder and deck was calculated based on the model provided in AASHTO LRFD Bridge Design Specifications (2020). Variation of modulus of elasticity with time was based on ACI 209R-92 model. For the girder, the coefficients α and β for ACI 209R-92 model were back-calculated using the modulus of elasticity at prestress release and 28 days as anchor points whereas for the deck, values that correspond to Type I cement and moist curing ($\alpha = 4, \beta = 0.85$) were considered. In addition, 51 sections were

considered along the beam half-span. When employing different creep and shrinkage models, Type III cement was selected for girder concrete, when asked. Moreover, when the predictions were obtained using various creep and shrinkage models and measured properties, the average measured concrete compressive strengths at prestress release and 28 days were used.

Figure 8.3 shows the variation of measured and predicted camber with time for all four beams that belongs to Tiffin Street Overpass, TSO, project for which camber data was recorded. The predicted camber growth curves were obtained based on various creep and shrinkage models. In Figure 8.3a, the measured camber data is compared with predicted camber based on creep coefficients and shrinkage strains obtained from all considered models. As can be seen, the Bazant Baweja B3 model and the GL2000 model significantly overestimate beam camber. Therefore, these two models are excluded in Figure 8.3b so that the difference between the other models can be assessed more clearly. AASHTO LRFD Specifications (2020) provide two sets of equations to account for the effect of volume-to-surface ratio (V/S). The first set of equations is provided in the body of AASHTO (2020) and the second in the commentary. Figure 8.3b suggests that for one out of 4 beams, AASHTO LRFD (2020) body provision leads to more accurate predictions when compared to other models. For two out of 4 beams, CEB MC90-99 model results in better match with the measured data than other creep and shrinkage models. Finally, for one out of 4 beams, fib MC 2010 model gives the best match with the measured camber values among provided 8 creep and shrinkage models.

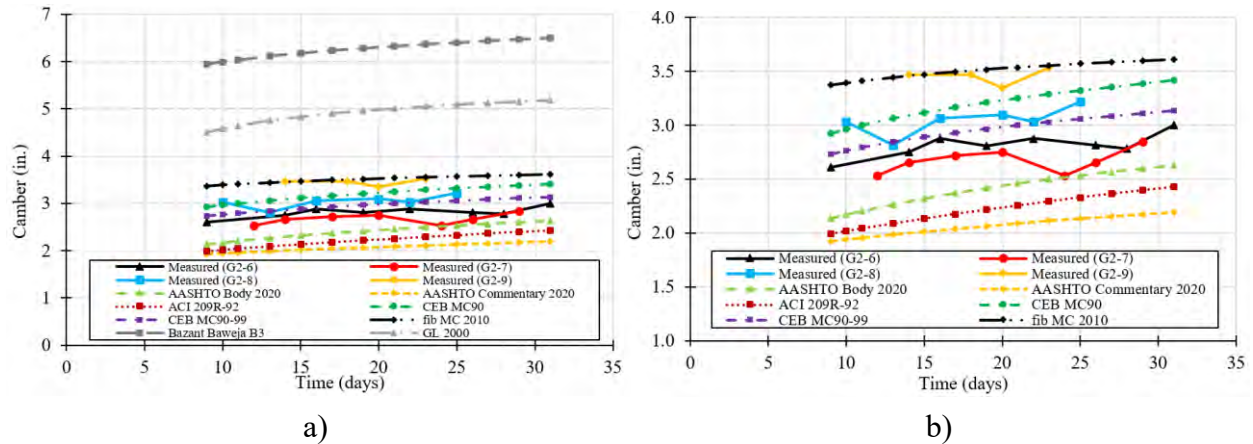


Figure 8.3. Variation of measured and predicted camber for the prestressed concrete beams fabricated in PSI Windsor for the Tiffin Street Overpass Project: a) full scale, b) reduced scale

In addition, Figure 8.4 shows the variation of measured and predicted camber with time for 14 beams whose data was collected by Hinkle (2006). All beams have identical cross-sectional properties, however, the span lengths for the Strand Pattern I and Strand Pattern II groups are 128 ft. and 136 ft., respectively. Therefore, the strand layouts for these two groups are different from each other. The predicted camber growth curves were obtained based on various creep and shrinkage models. In addition to the creep and shrinkage models, the analysis was conducted once using specified concrete properties, and another time using measured concrete properties. Since the Bazant Baweja B3 model and the GL2000 model significantly overestimated beam camber, they were not included in this analysis. Except these two models, all remaining creep and shrinkage models were considered.

Figure 8.4a and Figure 8.4b show the measured and predicted camber growth curves for Strand Pattern I beams (Hinkle 2006) using specified and measured concrete compressive strengths, respectively. Based on these figures, it can be said that overall using measured properties improved the predictions. It should be noted that although the beams have identical properties i.e. cross-sectional properties, mix designs and strand layouts, the measured cambers vary as much as 1.5 in. because of the inherent variability of concrete properties. Therefore, it is not possible to draw

a firm conclusion about which model gives the best predictions, however AASHTO LRFD 2020 model based on commentary provision and fib MC 2010 model provides the lower and upper bound of the predictions, respectively.

Figure 8.4c and Figure 8.4d show the measured and predicted camber growth curves for Strand Pattern II beams (Hinkle 2006) using specified and measured concrete compressive strengths, respectively. Similar to the predictions for first set of beams, it can be said that overall using measured properties improved the predictions (Figure 8.4c and Figure 8.4d). Due large scatter in the measured data although they have the same properties in terms of mix design, cross-sectional properties and strand layouts, it is not possible to draw a conclusion about which model provides the best match with the measured data. For this set of beams, however, AASHTO LRFD 2020 model based on commentary provision provides the lower bound of the predictions.

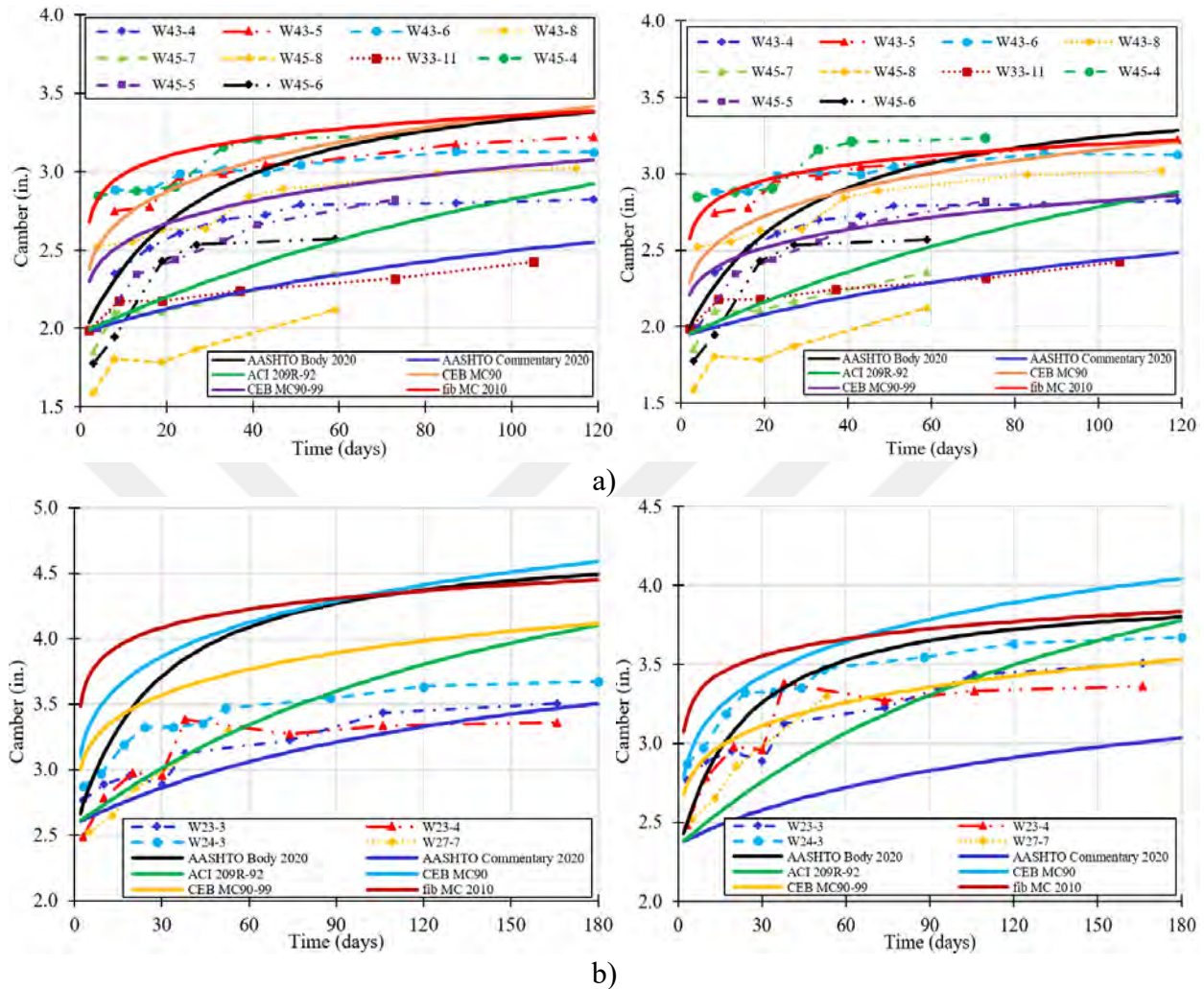


Figure 8.4. Variation of measured and predicted camber using specified (left) and measured (right) concrete properties for the prestressed concrete beams provided by Hinkle (2006): a) Strand pattern I and b) Strand pattern II

In addition, Figure 8.5a-f show a comparison of measured and predicted time-dependent flexural deformations for the six composite prestressed concrete beams, five of which (Figure 8.5a-e) was tested by Branson et al. (1970) and remaining one (Figure 8.5f) was tested by Tadros et al. (1977). Predictions were conducted using the AASHTO LRFD (2020) models for creep and shrinkage provided in the main body and the commentary only, and considering measured properties if made available. In general, the use of commentary provisions of AASHTO LRFD (2020) for creep and shrinkage resulted in more accurate predictions of pre-erection camber

whereas those provided in the main body resulted in more accurate predictions of flexural deformation in the composite system (i.e. after deck is placed).

Figure 8.5g-h show a comparison for the composite steel beams tested by Bradford and Gilbert (1991) (Figure 8.5g) and Fan et al. (2010) (Figure 8.5h). Similar to prestressed concrete beams, only AASHTO LRFD (2020) model based on both main and commentary provisions for creep and shrinkage was considered, and measured concrete compressive strength was utilized, if made available. Figure 8.5g and Figure 8.5h show that the creep and shrinkage model based on the commentary provision of AASHTO LRFD (2020) predicted flexural deformation in the steel composite system better than that based on body provision of AASHTO LRFD (2020).

Considering the high number of factors that influence delayed deformations in composite concrete beams, the presented comparisons indicate that the proposed prediction methodology can predict with good accuracy the flexural deformation history of composite prestressed concrete and steel beams.

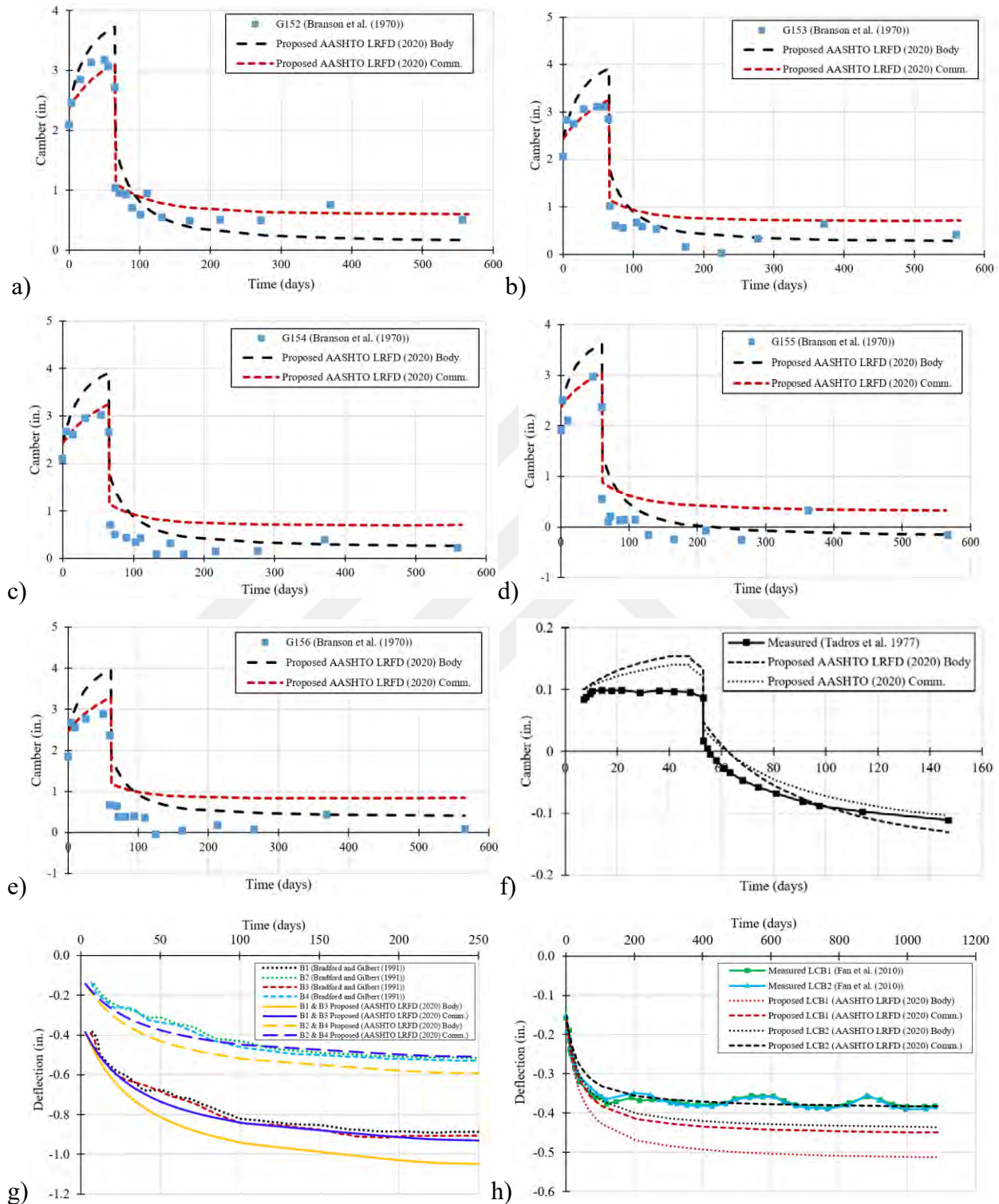


Figure 8.5. Comparison of measured and predicted time dependent flexural deformations: a) b) c) d) e) Periodic composite prestressed concrete beam flexural deformation data by Branson et al. (1970); f) Periodic composite prestressed concrete beam flexural deformation data by Tadros et al. (1977); g) Composite steel beam flexural deformation data by Bradford and Gilbert (1991) and h) composite steel beam flexural deformation data by Fan et al. (2010)

Moreover, Figure 8.6 shows the contribution of creep and shrinkage to the predicted long-term camber based on the AASHTO LRFD 2020 model. The difference in downward deflections caused by beam shrinkage based on the two sets of equations for V/S ratio is negligible, whereas the difference in camber caused by creep is noticeable. Additionally, the difference between predictions based on the provisions for V/S ratio provided in the body of AASHTO LRFD 2020 and those provided in the commentary are also noticeable. Since it has been shown in Table 8.2, Figure 8.3 and Figure 8.4 that the provisions in the body of AASHTO LRFD Bridge Design Specifications (2020) for creep and shrinkage led to reasonably accurate camber predictions compared to the other models, this model was used to evaluate the rest of the camber and displacement data. It should be noted, however, that this conclusion is based on the database used in this thesis. Menkulasi et al. (2018) arrived at a similar conclusion for three sets of prestressed concrete beams fabricated in Virginia. In addition, the AASHTO LRFD 2020 creep model provided the best match for the creep coefficient obtained based on measured creep data collected as part of this research project for one Michigan prestressed concrete beam mix (Figure 3.26). While the reasons stated above should provide a level of comfort in using the benchmark model set used in this research (i.e. AASHTO LRFD 2020 model for creep, shrinkage, and modulus at prestress release and 28 days, and ACI 209R-92 model for the variation of modulus of elasticity with time), it should be noted that other combinations of models could also result in accurate pre-erection camber predictions. ACI Committee 209 is perhaps in the best position to recommend a model for predicting creep and shrinkage. However, as stated earlier, the committee has refrained from doing so due to a lack of consensus regarding the data set and statistical indicators that should be used to arrive at such conclusion.

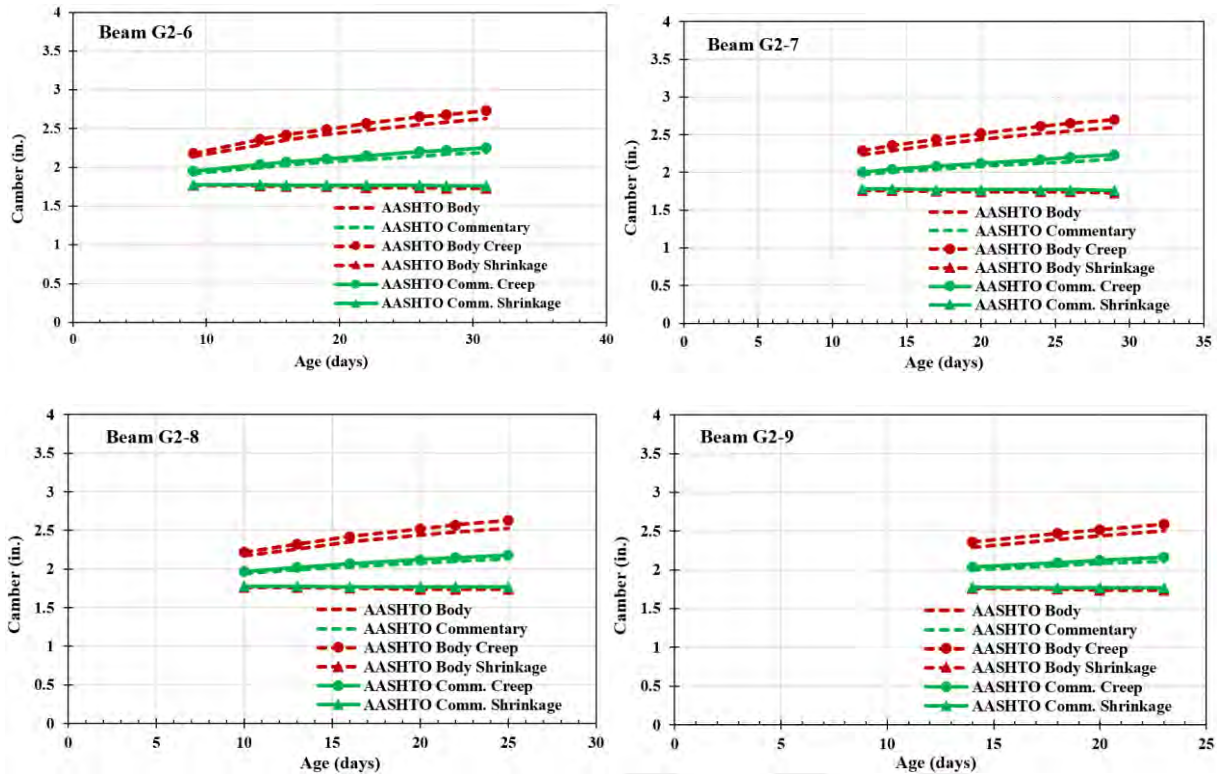


Figure 8.6. Influence of beam creep coefficient and beam shrinkage strain on beam camber considering the beams used in the Tiffin Street Overpass project

8.5. Comparison of Incremental Time Step Method (Nilson 1987) and Proposed Prediction Methodology

The Incremental Time Step Method (ITSM) (Nilson 1987) described in section 2.5.2 was used to compare pre-erection camber predictions with those obtained using the proposed prediction methodology. As noted in section 2.5.2, the ITSM (Nilson 1987) relies on updating the prestressing force for various time steps to capture the variation of this force with time so that the influence of this variation on beam camber and displacements can be quantified. The ITSM does not capture the direct effect that phenomena such as differential shrinkage between the deck and the girder can have on beam camber and displacement in cases where there is no prestress (such as a steel bridge). Therefore, this comparison was limited to pre-erection camber predictions in which the effect of differential shrinkage is limited compared to the condition after the deck is cast. Because the ITSM relies primarily on an accurate estimation of prestress losses to determine the variation of the

prestress force with time and consequently the variation of camber and displacements, a comparison between prestress losses computed using the AASHTO LRFD Bridge Design Specifications (2020) and those computed using the proposed methodology is conducted.

8.5.1. Variation of Prestress and Prestress Losses with Time

8.5.1.1. Variation of Total Prestressing Stress with Time

The variation of prestressing force and prestress losses with time was investigated to show the effect that certain construction activities such as deck placement or replacement have on the magnitude of the prestressing force and consequently on the magnitude of camber and displacements. This investigation was done using Approach No. 6 of the proposed prediction methodology. Figure 8.7 shows the variation of prestress with time for S11, B05, M57 and Shepherd Road-2 projects. The variation of the prestressing stress is shown at midspan. The variation of the total prestressing force over time is characterized by an initial gradual reduction due to time-dependent prestress losses caused by creep and shrinkage of beam concrete and relaxation of prestressing steel. When the deck is placed, there is a sudden increase in the prestressing force (prestress gain) due to the weight of the deck. Between deck placement and installation of superimposed dead loads (barrier + overlay), there is a minor gradual decrease due to time-dependent losses. The installation of barrier and overlay causes another increase in the prestressing force due to the induced tension. Then, there is a gradual decrease in the prestressing force due to the combined effects of creep of beam concrete, creep of deck concrete, differential shrinkage between beam and deck concrete, and differential shrinkage induced creep. When the deck is replaced (for example after 50 years), there is a sudden decrease in the prestressing force due to the rebound of the beam and the compression caused at the centroid of tendons because of this rebound. The sudden decrease in prestress (prestress loss) is recovered when the new deck is

placed. These observations are intuitive and are a confirmation that the proposed prediction methodology has the ability to quantify the variation of prestressing force with time and lead to logical results.

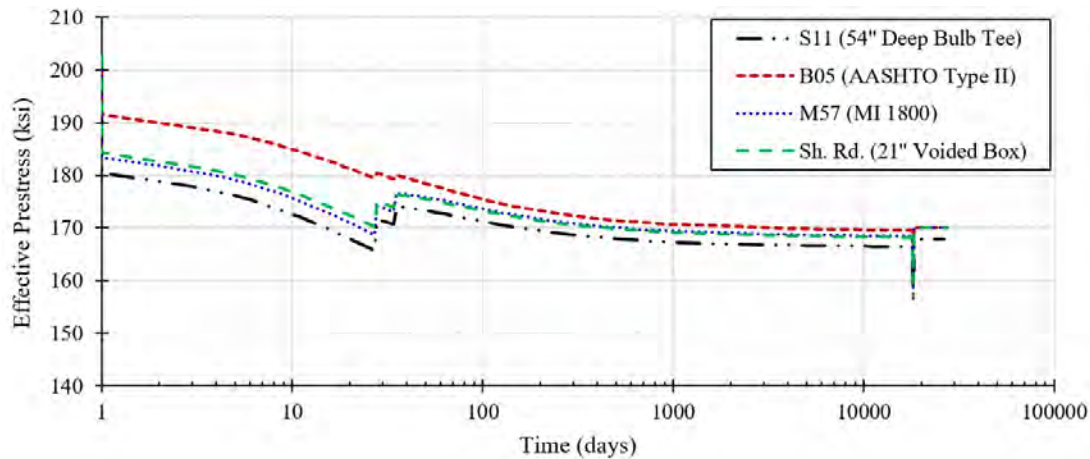


Figure 8.7. Variation of prestress as a function of time in a composite bridge system at midspan for S11, B05, M57 and Shepherd Road-2 projects

8.5.1.2. Variation of Prestress Losses with Time

There are various factors that affect the magnitude of the prestressing stress over time. To illustrate the effect of these factors on the prestressing stress, the prestress loss was plotted as function of time in Figure 8.8 for S11, B05, M57 and Shepherd Road-2 projects. A prestress loss is designated as positive, and a prestress gain is designated as negative. Figure 8.8 shows the variation of prestress loss or gain due creep and shrinkage of concrete and relaxation of steel as a function of time. The solid lines represent net effect that creep, shrinkage, and relaxation of steel have on prestress losses. The dashed lines represent the separate effects of girder and deck creep and girder and deck shrinkage. It should be noted that in reality these losses are interdependent (i.e. the loss due to shrinkage affects the loss due to creep, etc.). However, in this exercise, the variation of each loss with time was considered separately (i.e. when creep loss is calculated, the other losses are set equal to zero). As can be seen, concrete creep is the main contributor to prestress loss, followed by concrete shrinkage, and relaxation of steel. Girder creep causes the

majority of prestress loss a portion of which is counterbalanced by deck creep, which helps increase positive curvature when the deck is placed. Girder shrinkage is the second largest source of prestress loss, a portion of which is counter balanced by deck shrinkage, which causes positive curvature in the composite system thus causing a prestress gain. Finally, the relaxation of prestressing strands is also partly responsible for prestress losses although it has a lower influence compared to girder creep and shrinkage.

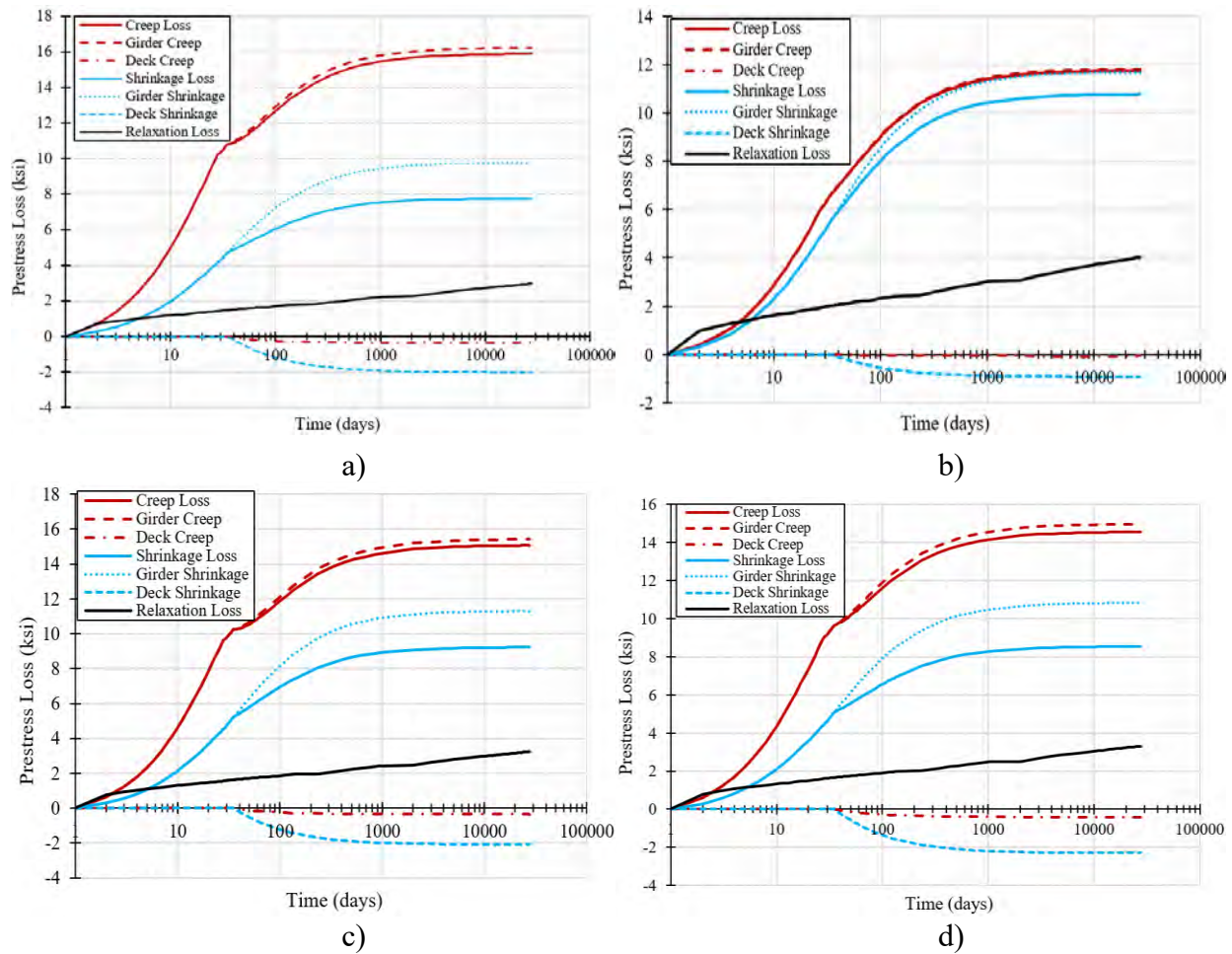


Figure 8.8. Variation of prestress losses with time in a composite bridge system at midspan for: a) S11 project; b) B05 project; c) M57 project and d) Shepherd Rd.-2 project

8.5.1.3. Comparison of Prestress Losses

An accurate prediction of prestress losses is important when estimating camber and displacements. The author believes that the proposed approach is accurate not only in terms of

predicting time-dependent camber and displacements but also in terms of predicting long terms losses. Recognizing that various states or consulting companies may use the ITSM (Nilson 1987), to predict time-dependent camber and displacements, the long-term losses computed based on the proposed method were compared with those obtained using AASHTO's body provisions (2020). An accurate prediction of prestress losses is also important when checking stresses at service.

AASHTO LRFD Bridge Design Specifications (2020) provide a refined method for estimating losses by considering two main time steps. The first covers the period from release of prestress to deck placement, and the second from deck placement to final time. Results obtained from AASHTO's (2020) refined method for calculating prestress losses were compared with those obtained from the proposed prediction methodology and are illustrated in Figure 8.9 and Table 8.3 for four projects: S11, B05, M57 and Shepherd Road-2. When using the proposed prediction methodology, total losses were once computed by ignoring the interdependence of losses and another time by considering this interdependence. The former approach was used to be able to compare prestress loss components causes by each phenomenon and the latter was used for a more realistic assessment. It should be noted that the proposed prediction methodology provides losses due to creep and shrinkage as a single numerical value since these phenomena are considered simultaneously. To separate these losses, the analysis was once run by setting the creep coefficient and relaxation loss equal to zero to obtain the shrinkage loss, and another time by setting the shrinkage strain and relaxation loss equal to zero to obtain the creep loss.

In general, predictions based on AASHTO's body provisions (2020) were consistent with those obtained from the proposed prediction methodology. The largest difference was exhibited by the creep loss. The difference in creep loss prediction was smaller between transfer and deck placement and became larger from deck placement to final time. This is expected because once

the deck is placed, creep loss is affected by girder and deck concrete creep, and the AASHTO (2020) model captures only the influence of girder concrete creep. Predictions for shrinkage loss were rather consistent throughout the lifetime of the bridge. AASHTO LRFD Bridge Design Specifications (2020) have provisions that capture prestress gains due to deck shrinkage after the deck is placed and for all considered projects, these provisions seem to be working well. Relaxation loss predictions were also consistent. A time-dependent function was used to predict losses due to relaxation of prestressing steel (Eqn. 3-119) in the proposed prediction methodology as opposed to the step-wise function recommended in AASHTO LRFD 2020. Table 8.3 shows the individual and total prestress losses as a % of jacking stress (f_{pj}) for the projects in question. The values inside the parentheses were obtained by considering the interdependence effect of prestress losses. For S11 project, the total time-dependent losses from release of prestress to deck placement based on the proposed prediction methodology were equal to 7.8% and 7.5% of the jacking stress when loss interdependence was ignored and considered, respectively, whereas those obtained from AASHTO Specifications (2020) were equal to 8.1%. Similarly, losses from deck placement to final time based on the proposed prediction methodology were equal to 5.3% and 4.4% whereas those based on AASHTO specifications (2020) were equal to 3.8%. When elastic shortening losses are considered total losses based on the proposed prediction methodology are equal to 24.0% and 22.8% and those based on AASHTO (2020) provisions are equal to 22.9% (Table 8.3). Overall, predictions based on AASHTO LRFD Bridge Design Specifications (2020) were rather accurate. When all project results are considered, the % difference for creep and shrinkage is no greater than 18%. Relaxation loss prediction differences are up to 68% although this is due to the small benchmark relaxation loss value. The maximum difference for total loss when interdependence is ignored does not exceed 34%. When interdependence is considered, the % difference for total loss

between proposed prediction methodology and AASHTO LRFD 2020 body provision does not exceed 33%.

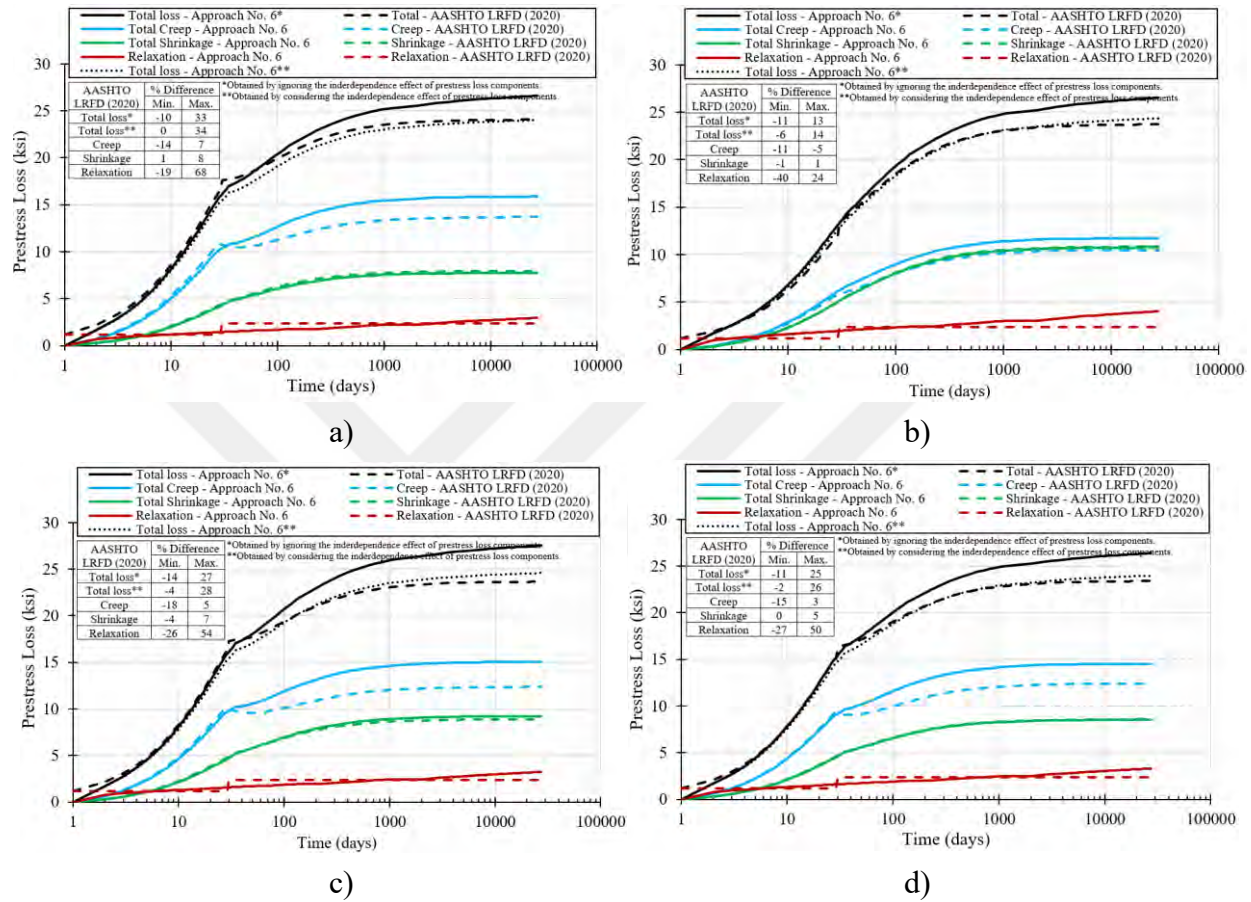


Figure 8.9. Comparison of calculated separate and total prestress losses using proposed prediction methodology and AASHTO LRFD 2020 Body provisions for: a) S11 project; b) B05 project; c) M57 project and d) Shepherd Rd.-2 project

Table 8.3. Comparison of separate and total prestress losses computed using the proposed prediction methodology and AASHTO LRFD 2020 Body provisions for the S11, B05, M57 and Shepherd Road-2 projects

Time Frame	Method	Project	ES* (%)	RL (%)	Creep (%)		Shrinkage (%)		Total time-dependent losses** (%)
					Due to girder	Due to deck	Due to girder	Due to deck	
From initial to deck placement	Approach No. 6	S11	10.9	0.7 (0.5 [†])	5.0	NA ⁺	2.0	NA ⁺	7.8 (7.5 [†])
		B05	5.4	1.0 (0.8 [†])	3.0	NA ⁺	2.4	NA ⁺	6.4 (6.1 [†])
		M57	9.4	0.8 (0.6 [†])	4.8	NA ⁺	2.2	NA ⁺	7.8 (7.5 [†])
		Sh. Rd.	9.0	0.8 (0.6 [†])	4.4	NA ⁺	2.2	NA ⁺	7.4 (7.1 [†])
	AASHTO LRFD [§] (2020)	S11	11.0	0.6	5.4	NA ⁺	2.1	NA ⁺	8.1
		B05	5.1	0.6	2.8	NA ⁺	2.4	NA ⁺	5.9
		M57	9.3	0.6	5.0	NA ⁺	2.3	NA ⁺	8.0
		Sh. Rd.	8.9	0.6	4.6	NA ⁺	2.2	NA ⁺	7.4
From deck placement to final	Approach No. 6	S11	NA ⁺	0.8 (0.3 [†])	3.0	-0.2	2.8	-1.0	5.3 (4.4 [†])
		B05	NA ⁺	1.0 (0.5 [†])	2.8	0.0	3.3	-0.5	6.7 (5.9 [†])
		M57	NA ⁺	0.8 (0.3 [†])	2.9	-0.2	3.3	-1.0	5.8 (4.7 [†])
		Sh. Rd.	NA ⁺	0.8 (0.4 [†])	2.9	-0.2	3.2	-1.1	5.6 (4.7 [†])
	AASHTO LRFD [§] (2020)	S11	NA ⁺	0.6	1.4	NA ⁺	2.2	-0.3	3.8
		B05	NA ⁺	0.6	2.3	NA ⁺	3.0	0.0	5.9
		M57	NA ⁺	0.6	1.1	NA ⁺	2.6	-0.6	3.7
		Sh. Rd.	NA ⁺	0.6	1.6	NA ⁺	2.5	-0.5	4.2
Total from initial to final	Approach No. 6	S11	24.0 (22.8 [†])						
		B05	18.5 (17.4 [†])						
		M57	23.0 (21.6 [†])						
		Sh. Rd.	22.0 (20.8 [†])						
	AASHTO LRFD [§] (2020)	S11	22.9						
		B05	16.9						
		M57	21.0						
		Sh. Rd.	20.4						

*Although it is shown in the table as a loss occurring between from initial to deck placement, it is an instantaneous loss, not time-dependent; **Combined time-dependent loss - elastic shortening is not included; [†]Not applicable; ES = Elastic Shortening; RL = Relaxation; [§]Body provision; [†]Predicted by considering interdependence of losses.

8.5.2. Comparison of Camber Prediction

The prestress losses computed using the AASHTO LRFD Bridge Design Specifications (2020) were used to update the prestressing force for several time steps with the purpose of computing pre-erection camber for S11, B05, M57, Shepherd Road-2, M5 and S10-2 projects using the Incremental Time Step Method (Nilson 1987) described in section 2.5.2. A total of 20 logarithmic like time steps were considered for all projects except M5 project where a total of 37 time steps were used when using Incremental Time Step Method (Nilson 1987) as well as the

proposed method. This means that the number of steps considered at the beginning of the considered time interval (i.e. between prestress transfer and pre-erection) was higher than that towards the end of the time interval to mimic the logarithmic like variation in creep and shrinkage of concrete. The results of this comparison are shown in Figure 8.10. In general, pre-erection camber predictions based on the proposed prediction methodology and Incremental Time Step Method (Nilson 1987) were similar with the proposed methodology providing better pre-erection camber for all projects where pre-erection camber measurements were made available. For general use, the proposed prediction methodology is recommended since it considers phenomena not accounted for by the Incremental Time Step Method (Nilson 1987) such as the influence of differential shrinkage between concrete and strands.

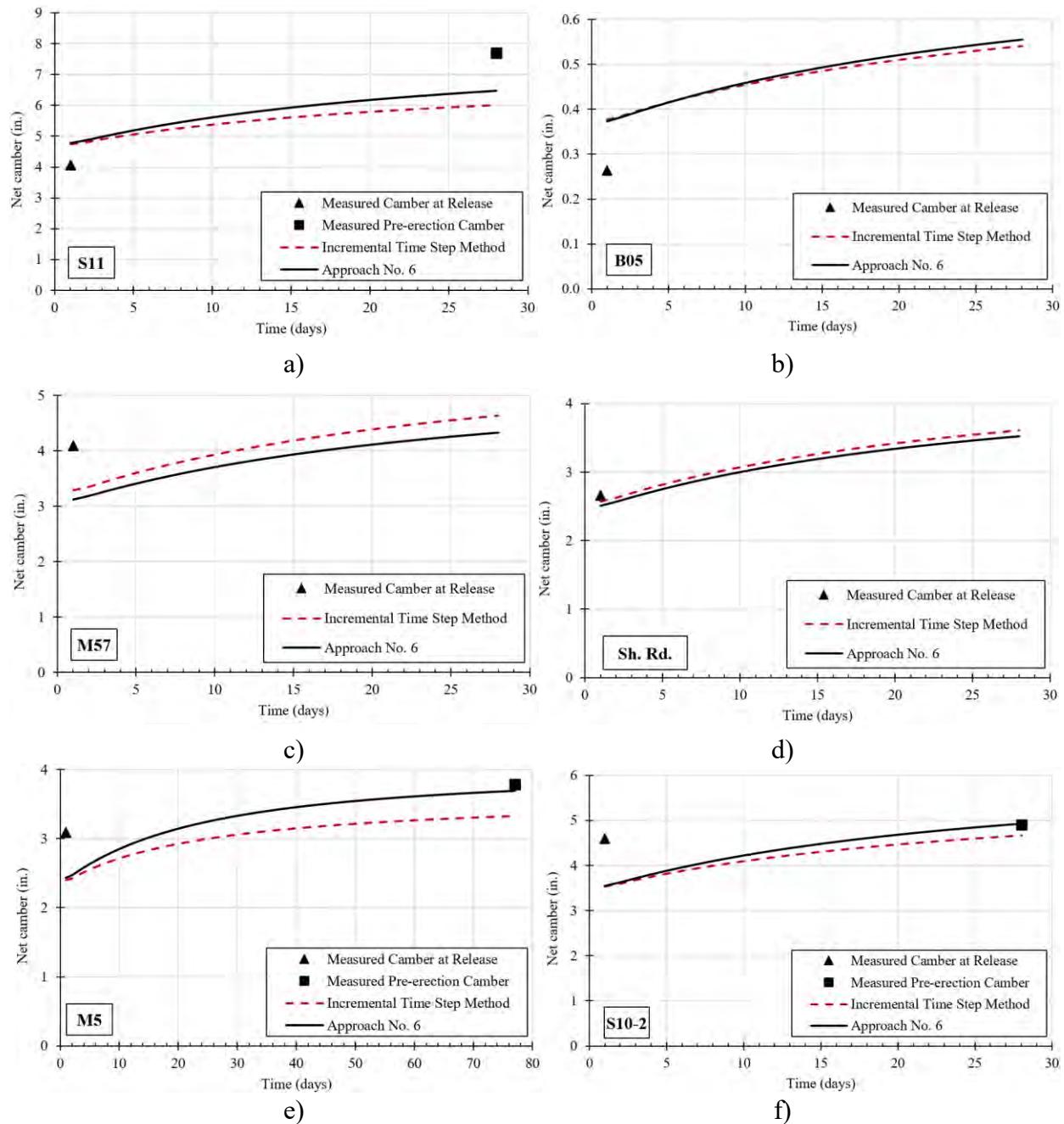


Figure 8.10. Comparison of proposed method with the Incremental Time Step Method (Nilson 1987) for the: a) S11 project; b) B05 project; c) M57 project; d) Shepherd Rd.-2 project; e) M5 project, and f) S10-2 project

8.6. Comparison of Various Approaches

Since the latest version of the proposed prediction methodology was based on several approaches each of introduced a level of theoretically enhanced accuracy, it is natural to ask what

the impact of the improvements introduced in each layer is on predictions of camber and displacements over time. This section provides a comparison of predictions conducted using all approaches and measured data, and contains three subsections. The first includes a comparison of all approaches for the displacement history of the beam up to pre-erection. The second includes the full-service life of the beam but excludes deck replacement. The third addresses the full displacement history including deck replacement. Unless otherwise stated, the predictions in this section were based on:

- AASHTO LRFD 2020 creep and shrinkage model with body provision (C.1.1 and C.2.1),
- AASHTO LRFD 2020 model for prediction of modulus of elasticity (3.6.1.1),
- ACI 209R-92 model with back-calculated parameters for modulus of elasticity prediction with respect to time (3.6.2.1),
- 51 sections along the beam half-span, and
- Specified concrete compressive strength with no overstrength factors.

8.6.1. Comparison of Approaches No. 1 through No. 6 for Pre-erection Camber

Camber growth up to pre-erection was predicted using all approaches for the prestressed concrete beams used in S11, B05, M57 and Shepherd Road projects. The beam type used in these projects are 54” deep bulb tee, AASHTO Type II, MI 1800 and 21” voided box, respectively. By exploring different beam types, it was aimed to highlight the differences between them using various methods i.e. Approaches No. 1-6 and Incremental Time Step Method, ITSM (Nilson 1987). The results are shown in Figure 8.11. Approaches No. 1 and No. 2 can provide predictions at discrete points and are shown by the green dots, which represent camber at release of prestress and pre-erection camber. The red dots show the measured camber at release and at pre-erection, if available. Approaches No. 3 through No. 6 can provide the full displacement history. Predictions

based on Approaches No. 1-6 are virtually identical for all considered projects with % differences being smaller than 4%. Approaches No. 1 and No. 2, despite their relative simplicity, lead to similar predictions of pre-erection camber with Approach No. 6, which is believed to be the most accurate. On the other hand, ITSM (Nilson 1987) predicts lower camber than all other approaches for S11 and B05 projects whereas for M57 and Shepherd Road-2 projects, it predicts higher camber than all other approaches. Although % difference for Approach No. 1-5 does not exceed 4%, it reaches almost 10% for ITSM.

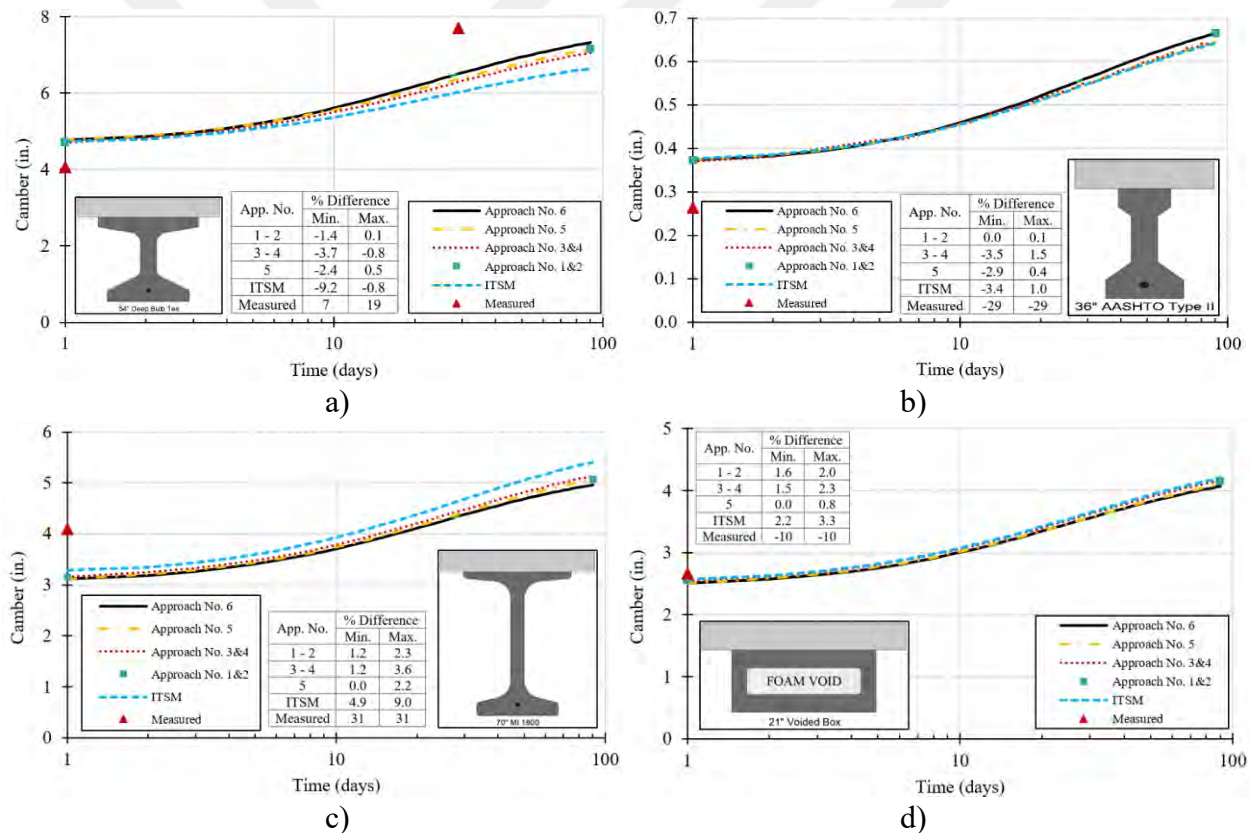


Figure 8.11. Comparison of various approaches for pre-erection camber for: a) S11 project, b) B05 project, c) M57 project and d) Shepherd Road-2 project

In addition, predictions based on Approaches No. 1 to No. 6 were evaluated using periodic camber measurements up to pre-erection. Analysis was conducted by utilizing both measured and specified concrete compressive strengths. When measured concrete strength was used, the average

of all beam measured concrete compressive strength values were considered. Results are shown in Figure 8.12. Figure 8.12a shows the measured camber measurements for 4 beams that belong to Tiffin Street Overpass project whose data was collected by research team and predictions based on 6 approaches. Based on this, all approaches from No. 1 to No. 6 underestimated the camber no matter which concrete properties were used. However, it should be noted that although these 4 beams had identical properties, the measured camber shows up to 1 in. variation. This may be explained by the inherent variability of concrete properties and temperature induced camber. Figure 8.12b-c show the periodic camber measurements that were collected by Hinkle (2006) and predictions based on Approaches No. 1 to No. 6. For the beams that belong to Strand Pattern I, using measured properties improved the predictions for all approaches. For the Strand Pattern II beams, however, using measured concrete compressive strengths did not improve the camber prediction for some of the approaches presented in this thesis. Due to large scatter in measured data, it is not possible to draw a conclusion about which approach together with which concrete properties i.e. measured or specified provides the best predictions. Since it involves all factors which affect prediction of camber and displacements, Approach No. 6 with measured concrete properties is suggested with abovementioned empirical models when predictions are obtained.

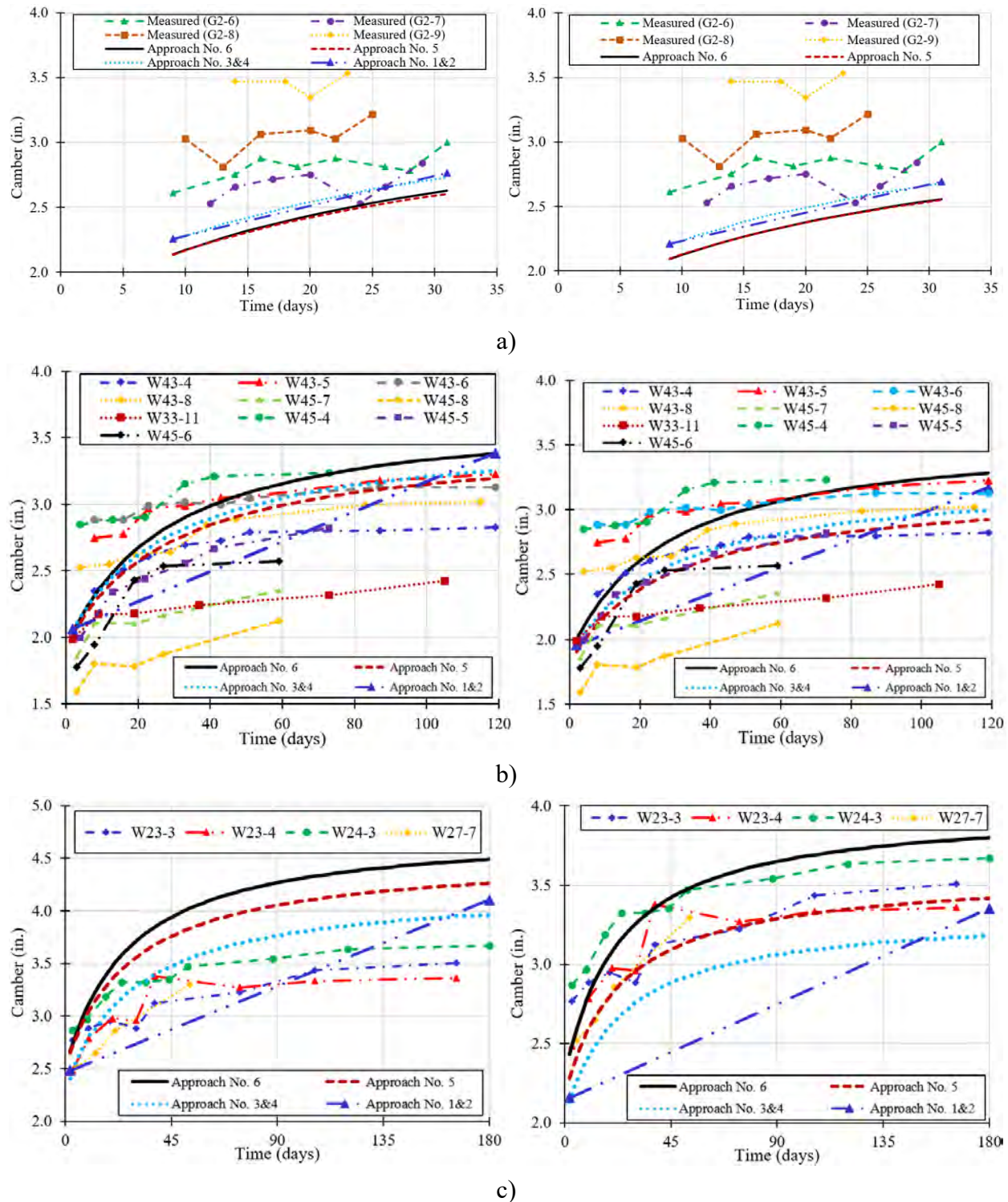


Figure 8.12. Comparison of various approaches using specified (left) and measured (right) concrete properties for: a) Tiffin Street Overpass Project, b) Cooper Road-1 Project (Hinkle 2006, Strand pattern I) and c) Cooper Road-2 Project (Hinkle 2006, Strand pattern II)

8.6.2. Comparison of Approaches No. 2 to No. 6 for Service Camber without Deck Replacement

Camber and displacements were predicted using Approaches No. 2 – 6 up to 75 years for the prestressed concrete beams used in the S11, B05, M57, Shepherd Road-2 and S10-2 projects, and steel beam used in M20 project. For the prestressed concrete beams, various beam types such as bulb tee, box, MI 1800 (Michigan specified beam), and AASHTO Type II were utilized. The results are shown in Figure 8.13. In this comparison, Approach No. 1 is not included because its scope was to provide predictions up to pre-erection and not beyond. All approaches result in rather similar predictions up to pre-erection for both prestressed concrete beam bridge projects (S11, B05, M57, Shepherd Road-2 and S10-2) with differences being less than 5%. For all considered projects, Approach No. 5 provides the highest service camber at 75 years. For S11 and M57 projects, Approach No. 6 predicts the lowest service camber at 75 years. Despite its simplicity, Approach No. 2 provides comparable results with other approaches. In addition, although there are fundamental differences between each of them, for S11, B05 and S10-2 projects, all approaches result in rather similar service camber at 75 years. However, for M57 project, the predictions based on Approach No. 6 are quite different from those obtained using other approaches. For S11, B05, Shepherd Road-2 and S10-2 projects, the difference between Approaches No. 2 and Approach No. 6 does not exceed 80%. For the same beams, the difference between Approach No. 3&4 and Approach No. 6 is at most 27%, and the difference between Approach No. 5 and Approach No. 6 is around 101%. Since the service camber at 200 days switches from upward to downward when Approach No. 6 is employed, the % difference results between Approaches No.2 – 5 and Approach No. 6 are very high for M57 project.

For the steel beam bridge project (M20), the difference between Approaches No. 3, 4 and 5 and Approach No. 6 is less than 9%, whereas the difference between Approach No. 2 and

Approach No. 6 is approximately 0.6%. It should be noted that in Approach No. 2 the placement of the deck and superimposed dead loads (barrier and overlay) is assumed to take place simultaneously whereas in Approaches No. 3 and above, a period of 7 days is assumed between the placement of the deck and placement of superimposed dead loads. As can be seen from Figure 8.13, the fundamental theoretical differences in each prediction methodology start to produce notable differences in camber and displacements at service in some cases.



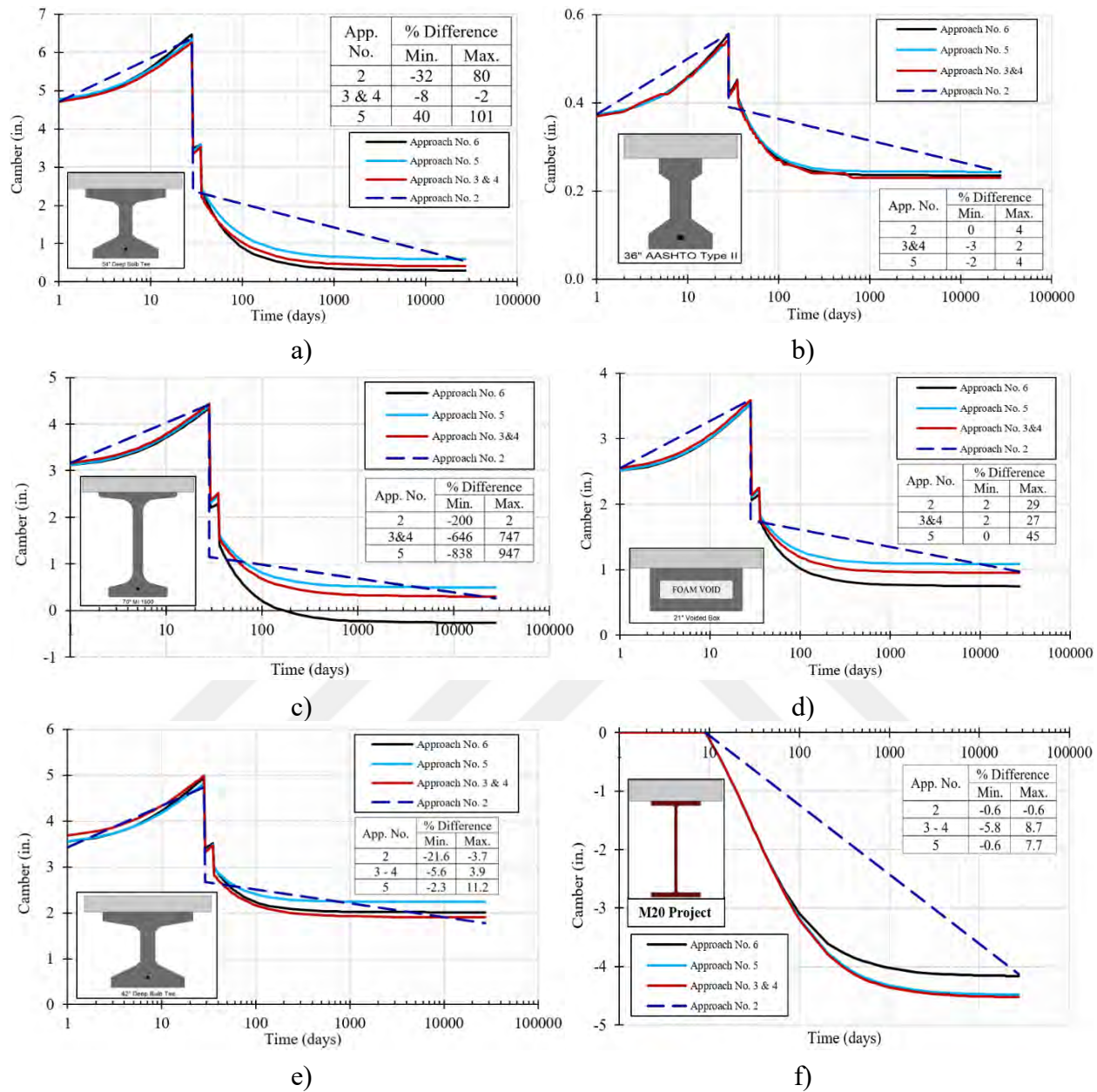


Figure 8.13. Comparison of predicted camber and displacement history based on Approaches No. 2 to No. 6 without deck replacement activity for: a) S11 project, b) B05 project, c) M57 project, d) Shepherd Road-2 project, e) S10-2 project and f) M20 project

In addition, predictions based on various approaches i.e. Approaches No. 2 – 6 were compared with the measured data provided by Branson et al. (1970), Tadros et al. (1977), Bradford and Gilbert (1991) and Fan et al. (2010) using measured concrete properties, if provided. Details

about these beams can be found in section 4.4, Table 4.3, or corresponding articles. The results are shown in Figure 8.14.

For the beams whose data was provided by Branson et al. (1970) in Figure 8.14a-e, Approach No. 5 and No. 6 overestimate the camber up to pre-erection whereas Approach No. 2 and No. 3&4 predicts the camber reasonably good up to pre-erection. For the post-erection zone, i.e. after deck is cast, 3 out of 5 beams, Approach No. 6 provides better match with the measured than all other approaches. In all cases, Approach No. 4 overestimates the service camber and provides the highest prediction when compared to other approaches. Despite its simplicity, Approach No. 2 predicts the service camber at the last day of measurements reasonably accurate. 2 out of 5 beams, Approach No. 5 provides the best predictions, although the difference between predictions based on Approach No. 5 and No. 6 is not significant.

For the beam which was tested by Tadros et al. (1977) in Figure 8.14f, there was no camber growth observed i.e. the camber stayed almost stable up to deck placement. However, predictions based on all approaches predicts some camber growth around 0.05 inches, therefore, they seem to be off up to pre-erection and Approach No. 2 provides the highest pre-erection camber and other approaches provide almost identical predictions with each other. For the post-erection zone, it seems Approach No. 2 provides the spot on prediction at the last day of measurements, however, this does not mean it is the best approach in this case because the error in pre-erection camber somehow leads this result. Other approaches, on the other hand, provide rather similar predictions at service. However, if the camber just after deck is placed and superimposed dead loads are applied is taken as reference point, it can be said that Approach No. 6 provides the best predictions when compared to other approaches.

Figure 8.14g-h show the results for the beams whose experiments were conducted by Bradford and Gilbert (1991). For the first of beams, B1 and B3 in Figure 8.14g, while Approach No. 6 provides the lowest predictions, Approaches No. 2 – 5 provides similar results with each other, and predict deflection better than Approach No. 6. For the second set of beams, B2 and B4 Figure 8.14h, on the other hand, Approach No. 6 provide the best match with the measured data when compared to other approaches, although the differences between the predictions are not significant.

For the beams reported by Fan et al. (2010) in Figure 8.14i-j, Approach No. 6 provides the best predictions among all other approaches. While the maximum difference between measured and predicted camber for LCB1 beam, Figure 8.14i, is at most 0.15 in., it is around 0.5 in. for LCB2 beam, Figure 8.14j, although the differences between predictions based on Approaches No. 2 – 6 are not significant.

Apparently, predictions based on various approaches do not vary significantly from each other when steel beam is considered. For the prestressed concrete beams, on the other hand, there may be some notable differences between approaches presented in this thesis. This may be expected because the number of parameters involving then prestressed concrete beams are considered are higher than those when steel beams are considered. It seems these parameters may affect the camber predictions when different approaches are utilized.

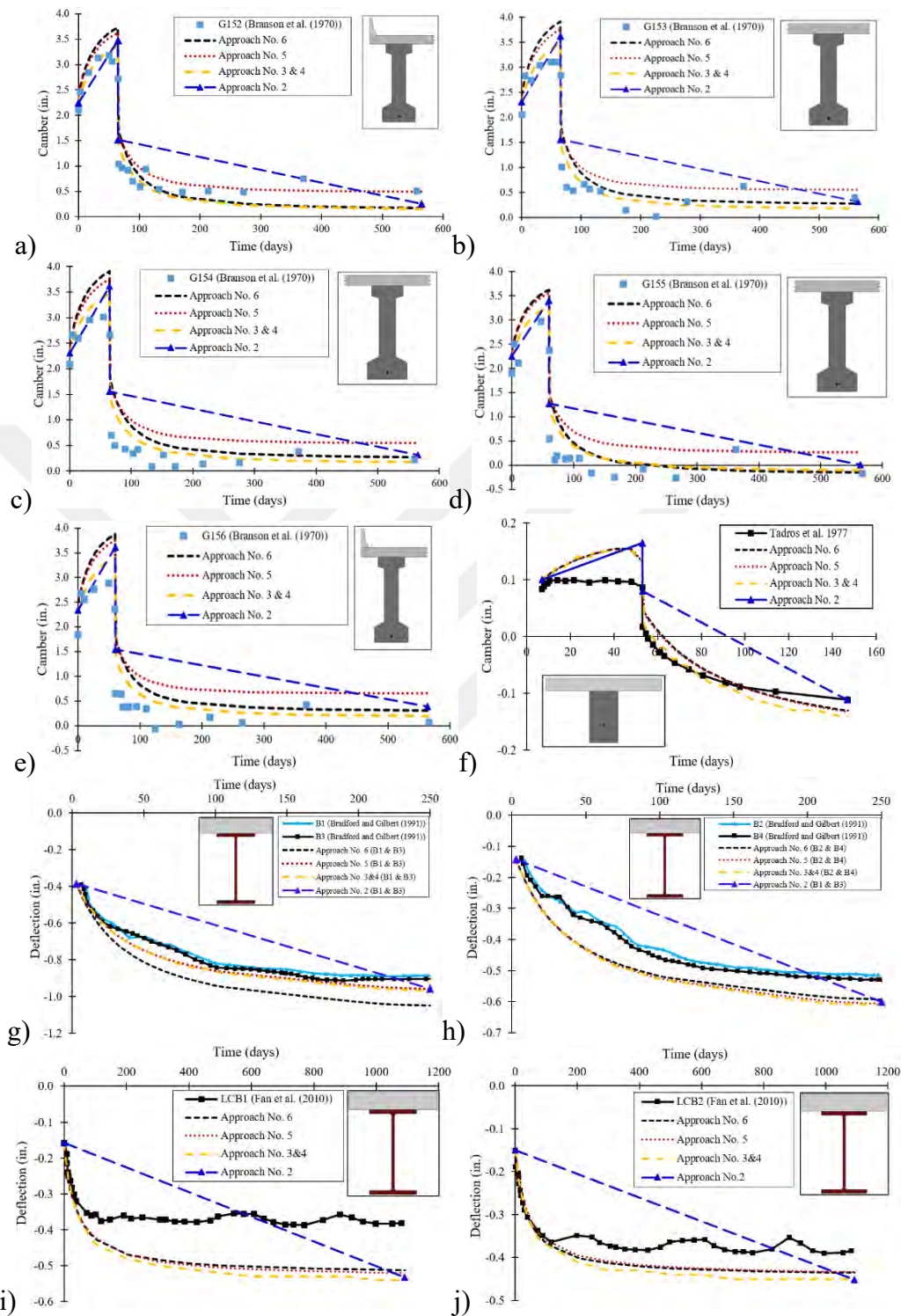


Figure 8.14. Comparison of measured and predicted time dependent flexural deformations using Approaches No. 1-6: a) b) c) d) e) Periodic composite prestressed concrete beam flexural deformation data by Branson et al. (1970); f) Periodic composite prestressed concrete beam flexural deformation data by Tadros et al. (1977); g) h) Composite steel beam flexural deformation data by Bradford and Gilbert (1991) and i) j) composite steel beam flexural deformation data by Fan et al. (2010)

8.6.3. Comparison of Approaches No. 4 to No. 6 for Service Camber with Deck Replacement

The full displacement history of the prestressed concrete beams used in the S11, B05, M57, Shepherd Road-2 and S10-2 projects was predicted using Approaches No. 4 – 6. A similar comparison was conducted for the steel beams in the M20 project. Two initial deck placement times were considered: 1) at 28 days, and 2) 90 days. This comparison includes an assumed deck replacement activity after 50 years from fabrication of beams. Approaches No. 1, 2, and 3 were not included in this comparison because their scope was limited either to the prediction of pre-erection camber (Approach No. 1) or the prediction of service camber without a deck replacement activity (Approaches No. 2 and No. 3). The results are shown in Figure 8.15 in terms of the full displacement history. Figure 8.15 suggests that there are notable differences in prediction at service between approaches when the initial deck is placed at 28 days after the factorization of the beam. These differences are more pronounced when initial deck placement time is raised up to 90 days. It is interesting to note that although the beam stress histories are significantly different for the cases when initial deck placement times are 28 and 90 days, no matter when the initial deck is placed i.e. either 28 or 90 days, the Approach No. 6 provides almost identical service camber predictions at 75 years. For the M20 project which featured steel beams the difference between camber and displacements did not exceed 11% and all approaches resulted in rather similar predictions. The results presented so far suggest that the difference in predictions based on various approaches is more pronounced in prestressed concrete beam bridge projects compared to steel beam bridge projects. This is due to the fact that any uncertainties related to the creep and shrinkage of the beam are removed when the beam is a steel beam since a steel beam does not creep or shrink. The fundamental differences between the various iterations are related for how to best predict the structural effects of creep in a prestressed concrete beam, which is subject to a varying stress history.

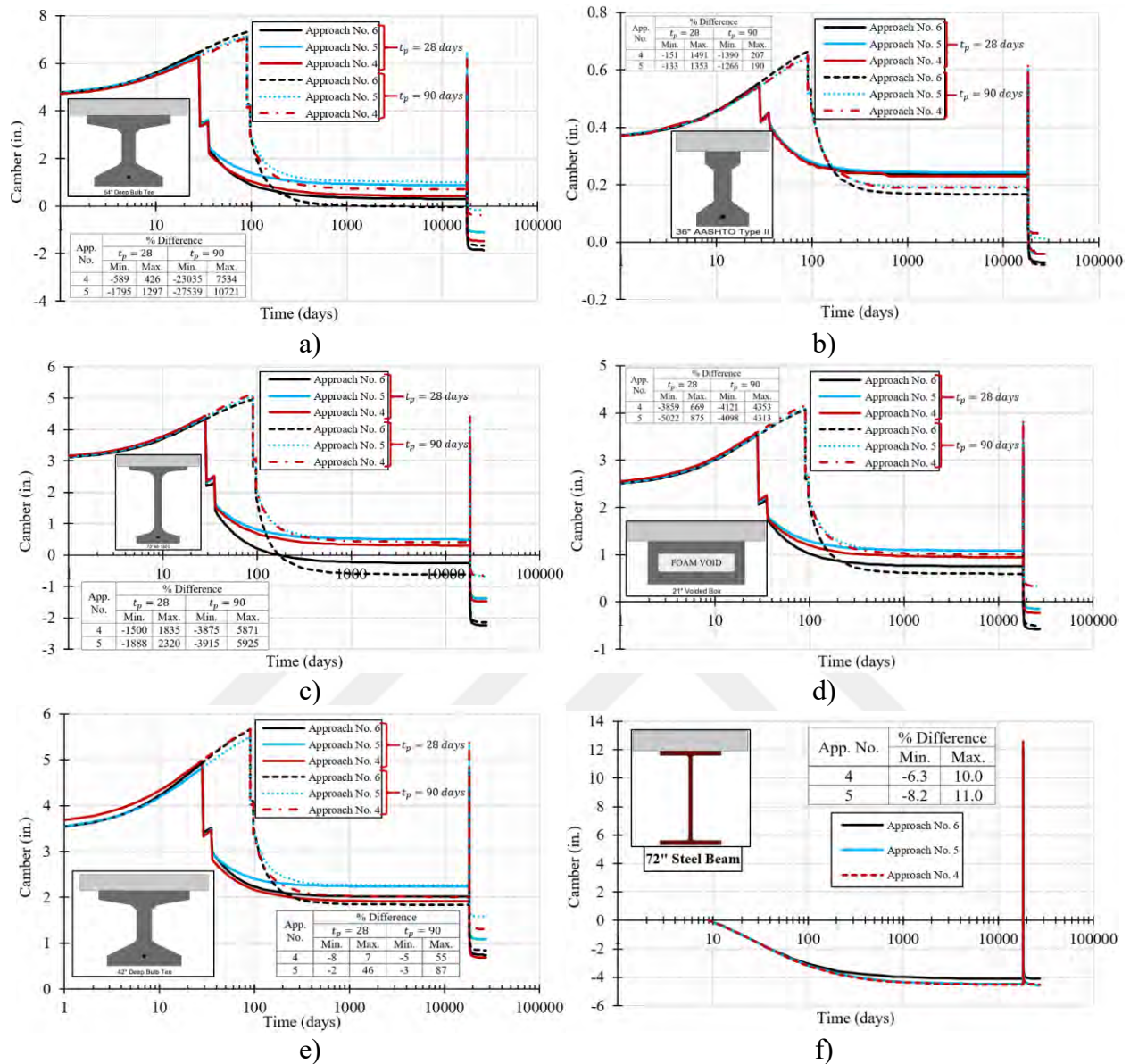


Figure 8.15. Comparison of predicted displacement history based on Iterations No. 4, 5, and 6 including a deck replacement activity after 50 years of service for: a) S11 project, b) B05 project, c) M57 project, d) Shepherd Road-2 project, e) S10-2 project, and f) M20 project

8.6.4. Comparison of Approaches No. 1 to No. 6 for Stresses

Although stresses are not directly related with the prediction of camber and displacements in bridge girders, a comparison study about effective prestressing strand stresses, concrete top and bottom fiber stresses and stresses along the cross-section of composite prestressed concrete beams using various approaches and various beam types is conducted to further elaborate the differences

between various approaches and to show the capabilities of proposed prediction methodology. *MDOTCamber* program can be used to obtain abovementioned stress results based on Approach No. 6 as illustrated in APPENDIX D. Unless otherwise stated, the predictions in this section were based on:

- AASHTO LRFD 2020 creep and shrinkage model with body provision (C.1.1 and C.2.1),
- AASHTO LRFD 2020 model for prediction of modulus of elasticity (3.6.1.1),
- ACI 209R-92 model with back-calculated parameters for modulus of elasticity prediction with respect to time (3.6.2.1),
- 51 sections along the beam half-span,
- Specified concrete compressive strength with no overstrength factors,
- First deck is placed at 28 days after beam fabrication, and
- First deck is removed at 50 years after beam fabrication, and second is placed 10 days after the deck removal event.

8.6.4.1. Comparison of Effective Prestressing Strand Stress using Approaches No. 4 to No. 6

Prestressing strand stress histories were compared using Approaches No. 4 – 6 for S11, B05, M57 and Shepherd Road-2 projects. The variation of prestressing stress is shown at midspan. Results are shown in Figure 8.16. As indicated in section 8.5.1.1, for all considered projects, the variation of total prestressing force over time is featured by an initial reduction due to elastic shortening loss, then further gradual reduction due to time-dependent losses i.e. creep and shrinkage of concrete and relaxation of prestressing strands. A sudden increase in stress is observed due to placement of deck, following by a gradual reduction due to time-dependent effects up to placement of overlay and barrier loads. An elastic gain is observed due to application of overlay and barrier loads and stress continues to decrease up to deck removal event. An elastic loss occurs

due to deck removal, and stress keeps decreasing until the new deck is placed due to time-dependent effects (not seen due to scale). A sudden increase in stress in the prestressing strand is observed due to placement of new deck. Then it decreases slightly until the placement of new barrier and overlay loads. An elastic gain is observed in the strand stress due to placement of such loads and it continues decreasing slightly until the end of service life of the girder. Since almost all of the creep and shrinkage of concrete and relaxation of prestressing strands have taken place, the decrease in strands stress after 50 years due to time-dependent effects is minor when compared to the losses after placement of first deck and application of superimposed dead loads.

Based on the results presented in Figure 8.16, it can be said that up to deck placement, all approaches provide rather similar results. After deck is placed and composite action starts, some differences between approaches are observed. For S11 and B05 projects, while the % difference in the strand stress between Approach No. 4 and No. 5, and Approach No. 6 does not exceed 0.80%, it reaches almost 5% for M57 and Shepherd Road-2 projects. These statistics suggest that despite the fundamental differences between Approaches No. 4 – 6, prestressing stress predictions does not vary significantly when different approaches are employed.

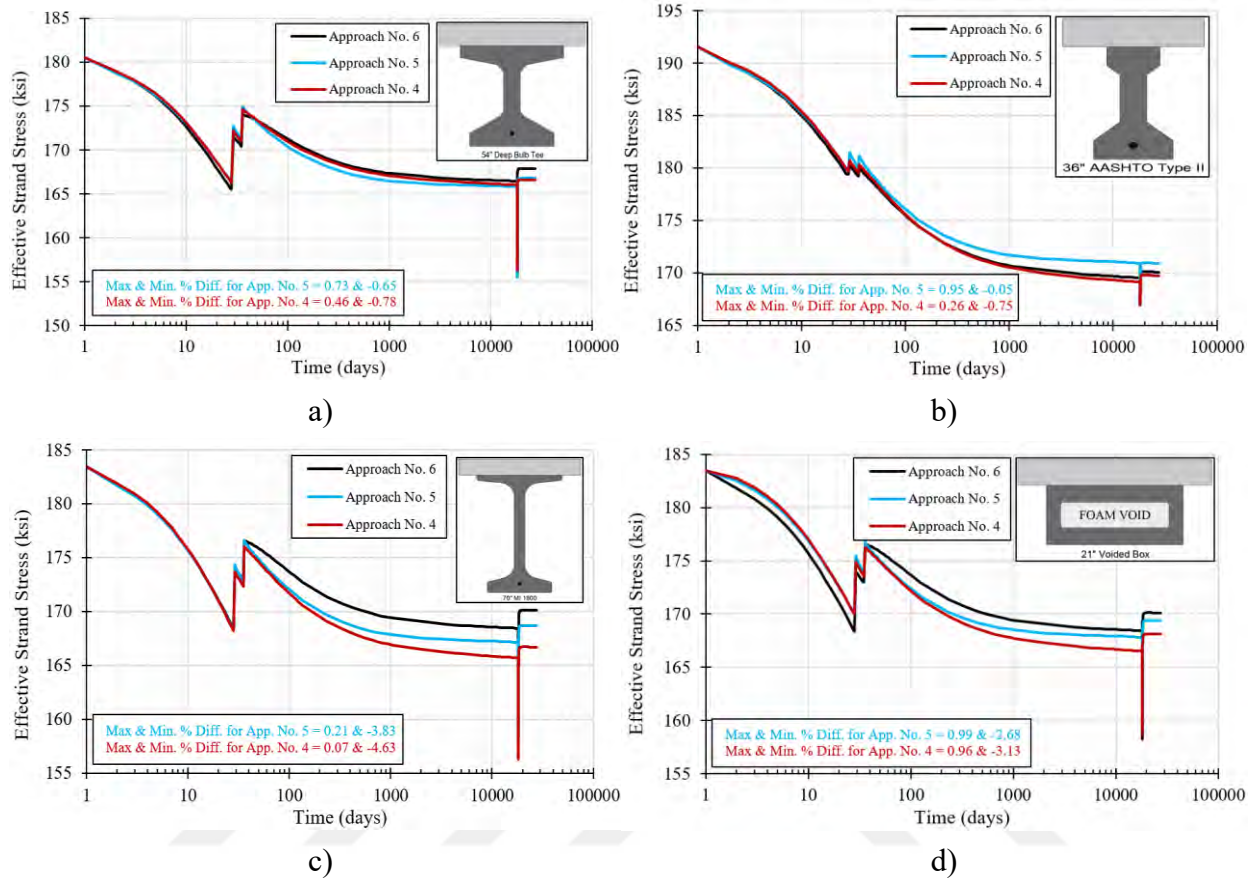


Figure 8.16. Comparison of strand stress histories using Approaches No. 4 to No. 6 for: a) S11 project, b) B05 project, c) M57 project, and d) Shepherd Road-2 project

8.6.4.2. Comparison of Top and Bottom Fiber Stresses using Approaches No. 4 to No. 6

Stresses at the top and bottom fibers of the prestressed concrete beams used in S11, B05, M57 and Shepherd Road-2 projects were compared using Approaches No. 4 – 6 and allowable stresses based on AASHTO LRFD Bridge Design Specifications (2020). In addition, stresses at the top and bottom fibers for the decks, first and second i.e. replaced, used in such projects were compared based on various approaches and cracking strength based on AASHTO LRFD 2020. The variation of stresses is shown at midspan based on sign convention stated in Figure 3.1. Results are shown in Figure 8.17 for prestressed concrete beams, and in Figure 8.18 for the decks used in these projects.

According to AASHTO LRFD 2020 section 5.9.2, allowable stresses shown in Eqns. 8-1 and 8-2 are used when comparing the concrete stresses with predicted ones.

$$\sigma_{transfer_{allowable}} = 0.65 \cdot f'_{ci} \quad 8-1$$

$$\sigma_{service_{allowable}} = 0.45 \cdot f'_c \quad 8-2$$

Where:

$\sigma_{transfer_{allowable}}$ is allowable stress at prestress transfer,

$\sigma_{service_{allowable}}$ is the allowable stress at service,

f'_{ci} and f'_c are the concrete compressive strengths at release and at 28 days, respectively.

Figure 8.17 shows that the predictions based on Approaches No. 4 – 6 are similar up to placement of deck. After deck is placed and composite action starts, some differences between Approaches No. 4 – 5 and Approach No. 6 are observed such that for the bottom fiber, Approach No. 6 provides slightly lower stresses whereas for the top fiber, Approach No. 6 provides slightly higher stresses. For these projects, beams are fully prestressed meaning that no tensile forces are developed throughout the service life based on all approaches. When top and bottom fiber stresses are compared with the allowable limits stated in Eqns. 8-1 and 8-2, it can be said that all considered beams are in the limits up to deck replacement based on all approaches. For S11, M57 and Shepherd Road-2 projects, however, stress at the top fibers exceeds the allowable limits after new deck is placed for same cases. While predictions based on Approaches No. 4 – 5 are above the allowable limits for S11, M57 and Shepherd Road-2 projects, Approach No. 6 predictions exceed the allowable limits for only M57 project. The reason why stresses at the top fiber increases after new deck is placed is explained by the fact that when the new deck is placed, which is assumed to be 50 years after beam fabrication, almost all the beam shrinkage has taken place whereas newly

placed deck has just been cast. Due to differential shrinkage between new deck and beam, the stresses at the top fiber increases.

For all considered projects, the variation of stresses at the top fibers over time is featured by gradual increase up to pre-erection due to time-dependent effects between prestressing transfer and deck placement. A sudden increase in stress at the top fiber is observed due to placement of initial deck. Since it is assumed that deck is moist-cured for 7 days, the shrinkage of initial deck does not take place in the first 7 days after composite action starts. The stress at the top fiber decreases slightly until the end of curing period and a sudden increase in the stress at the top fiber is observed due to application of superimposed dead loads (barrier and overlay). After curing period for the initial deck ends, deck starts to shrink, and due to differential shrinkage between initial deck and girder and prestress losses, stress at the top fiber continues to increase until the removal of initial deck. A sudden decrease at the top fiber stress is observed when the initial deck is removed. By the time the new deck is cast, while there is no change in stress at the top fiber when Approach No. 4 and 5 were used, there is minor decrease observed when Approach No. 6 is employed (could not be seen in the figures due to scale). This is mainly because of the employing distinct creep curves for each force element in the force history matrices i.e. axial and bending when Approach No. 6 is used. After new deck is placed, a sudden increase in top fiber stress is observed. It is noticed that in all approaches, the stress at the top fiber just after the new deck is placed is higher than the one just after the initial deck is placed. This is expected because during the period between initial and new deck placements, girder continues creeping and shrinking, and this causes time-dependent prestress losses, and consequently leads to increase in top fiber stress. After the new deck is cast, it is assumed that it subjects to moist curing for 7 days similar to initial deck. It should be noted that during the curing period of the new deck, while there is no change in

top fiber stress in girder when Approaches No. 4 – 5 are used, some small gradual decrease in stress is observed when Approach No. 6 is used. It should be recalled that while all approaches predict some decrease in stress at the top fiber for the initial deck, only Approach No. 6 predicts some minor decrease in the top fiber stress for the new deck during curing period. This is because of the advanced algorithm of the Approach No. 6 such that separate creep curves were used when predicting time-dependent flexural deformations and stresses. That is the change in forces i.e. axial and bending in the girder when the initial deck is removed is considered as a separate forces and corresponding creep curve is used, which is not the case for in all other approaches. Following the end of curing period for the new deck, superimposed dead loads are applied on the new composite structure which results in sudden increase in the stress at the top fiber. Similar to the reason in the initial deck, due to differential shrinkage between new deck and girder, and remaining time-dependent prestress losses, the stress at the top fiber keeps increasing and reaches almost constant value towards the end of service life of the bridge, which is assumed to be 75 years in the analysis. Similar conclusions can also be drawn for the stresses at the bottom fiber.

The difference between Approach No. 4 and Approach No. 6 for these four projects throughout the service life is at most 159% and 70% at the top and bottom fibers, respectively. The difference between Approach No. 5 and Approach No. 6 for these four projects throughout the service life does not exceed 66% and 66% at the top and bottom fibers, respectively.

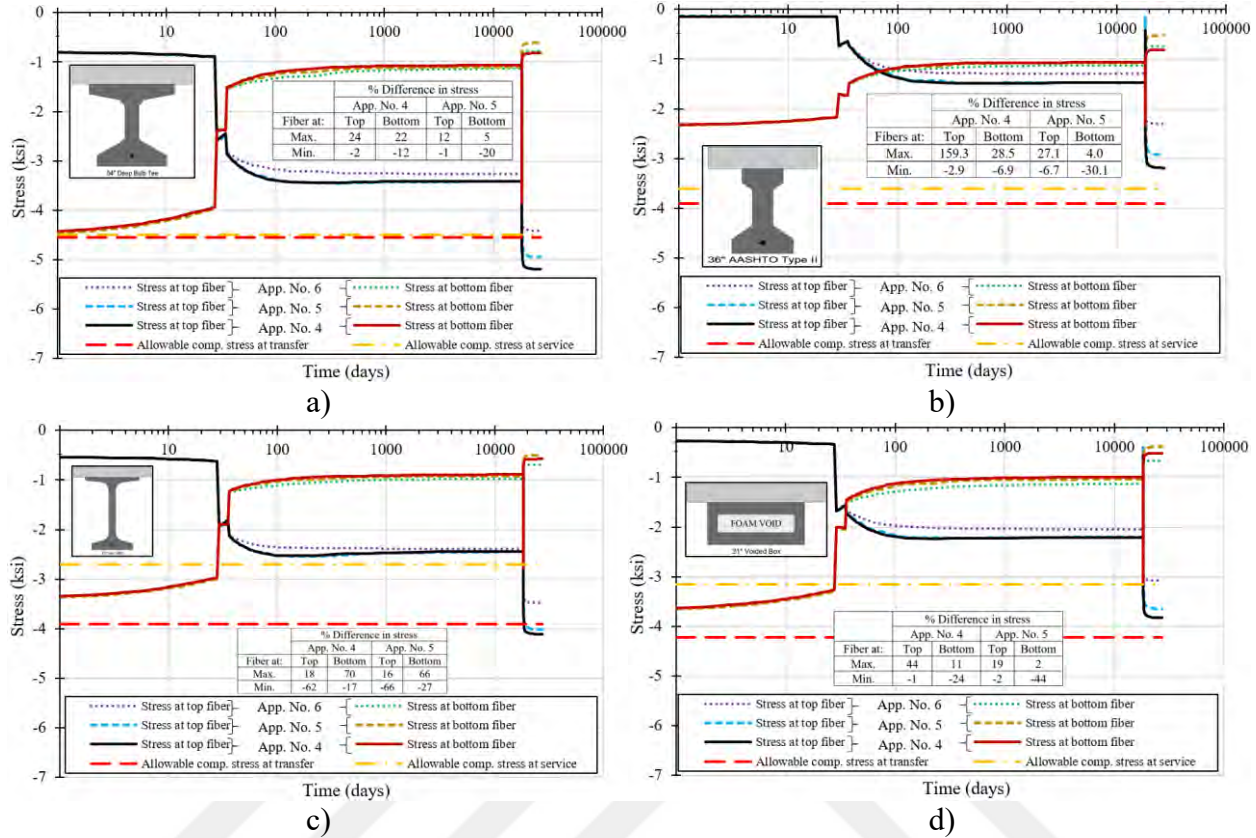


Figure 8.17. Comparison of top and bottom fiber stress histories for the beams used in: a) S11 project, b) B05 project, c) M57 project, and d) Shepherd Road-2 project

A similar study is also conducted for decks used in these four prestressed concrete beam projects. Results are shown in Figure 8.18. While comparing the top and bottom fiber stresses for decks with the cracking strength, Eqn. 8-3 is used in accordance with AASHTO LRFD 2020.

$$\sigma_{td} = 0.23\sqrt{f'_{cd}} \quad 8-3$$

Where:

σ_{td} is tensile strength for deck concrete,

f'_{cd} is the deck concrete compressive strength, ksi.

For S11 and M57 projects, the initial deck seems to be under compression until it is removed when various approaches are used (Figure 8.18a and Figure 8.18c). On the other hand,

for B05 project, tensile stresses are developed when Approaches No. 4 – 6 are used, but they do not exceed the cracking strength (Figure 8.18b). For Shepherd Road-2 project, while initial deck is always under compression based on Approach No. 6, minor tensile stresses are predicted based on Approaches No. 4 – 5 (Figure 8.18d). Stresses in the initial deck start developing after it is cast. After curing period ends which is assumed to be 7 days, superimposed dead loads are applied to the composite structure, and effects of them on the initial deck are calculated based on relative stiffness (axial and flexural) of the deck with respect to composite structure. Due to application of superimposed dead loads (barrier and overlay), a sudden increase in the top fiber stress for the initial deck is observed. Since the deck starts shrinking, and due to differential shrinkage between deck and girder, the stress in the deck decreases gradually. It is interesting to note that after some time the initial deck is cast, for the S11 and M57 projects, the stress in the top and bottom fibers starts increasing slightly when Approaches No. 4 – 5 are used, although this is not observed when Approach No. 6 is used. This is because of the fundamental differences between proposed approach (Approach No. 6) and other ones. While the forces are updated and single creep curve is used for the Approaches No. 4 – 5, each change in forces i.e. axial and bending are considered by using corresponding creep curves for Approach No. 6. This fundamental difference causes somehow mentioned effect in the stress history for the initial deck. At 50 years, initial deck is removed and, 10 days after it is removed, a new deck is placed. For all projects, after some time the new deck is cast, tensile stresses are developed in the new deck, which are not the case for the initial deck for S11 and M57 projects. This is mainly because the effect of differential shrinkage between girder and the new deck is more significant than the one between the girder and initial deck. As shown in Figure 3.22, most of the beam shrinkage takes place 50 years after it fabricated. When the new deck is placed, it tries to get shortened due to shrinkage, but girder tries to prevent

it, and this leads to develop tensile stresses in the new deck. When Approaches No. 4 – 5 are used, while stresses at the top fibers on the new deck for S11, M57 and Shepherd Road-2 projects exceed the cracking strength, for B05 project, it does not. When Approach No. 6 is employed, on the other hand, the tensile stresses for the new deck does not exceed the cracking strength for all considered projects. While Approaches No. 4 – 5 give similar results, Approach No. 6 predicts slightly higher stresses than other approaches when the stresses are compressive, and predicts slightly lower stresses than other approaches when the stresses are tensile. Since the stresses in the decks are small when compared to stresses in the girder, % differences between Approaches No. 4 – 5 and Approach No. 6 are quite high. The difference between Approach No. 4 and Approach No. 6 for these four projects considering both decks is at most 8065% and 1803% at the top and bottom fibers, respectively. The difference between Approach No. 5 and Approach No. 6 for these four projects considering both decks does not exceed 7063% and 1354% at the top and bottom fibers, respectively.

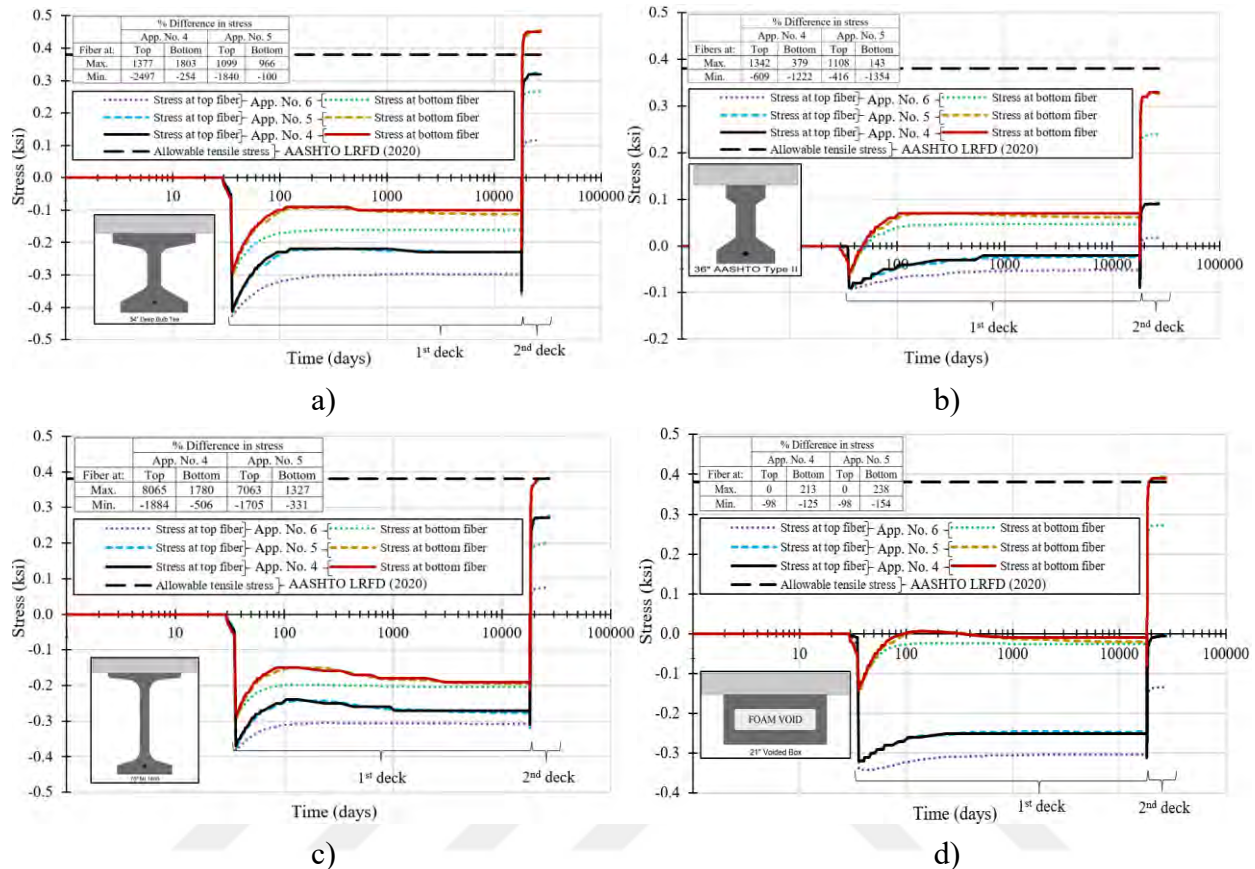


Figure 8.18. Comparison of top and bottom fiber stress histories for the decks used in: a) S11 project, b) B05 project, c) M57 project, and d) Shepherd Road-2 project

8.6.4.3. Comparison of Cross-sectional Stresses using Approaches No. 2 to No. 6

Stresses along the composite cross-section for four composite prestressed concrete beams used in S11, B05, M57 and Shepherd Road-2 projects were evaluated using Approaches No. 1 – 6 and allowable stresses and tensile strengths based on AASHTO LRFD 2020. It was considered in this section as well as throughout the analysis results presented in this thesis that plane section remains plane and there is a perfect bond between deck and girder. Results are shown in Figure 8.19. Analysis was conducted once at 100 days (Figure 8.19 left) and another time at 75 years (Figure 8.19 right) to involve stresses for both decks i.e. initial and replaced. For the prestressed concrete beams used in these four projects, the stresses along the cross-section are compressive stresses and they are within the allowable limits at 100 days when all approaches are utilized.

Despite the simplicity of Approach No. 2, the % difference between Approach No. 2 and Approach No. 6 is at most 8%. In addition, although there are fundamental differences between them, the % differences between Approach No. 3&4 and Approach No. 5, and Approach No. 6 do not exceed 14% and 7%, respectively. Similarly, for the decks used in S11, M57 and Shepherd Road-2 projects, the stresses along the cross-section are compressive stresses at 100 days when all approaches are used. For the B05 project, however, some minor tensile stresses are observed, but they do not exceed the cracking strength. The % difference between Approach No. 2 and Approach No. 6 is at most 30%. The % differences between Approach No. 3&4 and Approach No. 5, and Approach No. 6 do not exceed 107% and 94%, respectively.

When the analysis is conducted at 75 years, for the prestressed concrete beams used in these four projects, the stresses along the cross-section are compressive stresses, however they exceed the allowable limits stated by AASHTO LRFD 2020 for some cases. While Approaches No. 4 – 5 predicts stresses above the limits for S11 and Shepherd Road-2 projects, Approach No. 6 predicts the stresses along the cross-section within the allowable limits. For M57 projects, all approaches provide stresses above the limit. For B05 project, on the other hand, the stress predictions based on all approaches are within the allowable limits. For the stresses along the cross-section of the prestressed concrete beams, the % difference between Approach No. 4 and Approach No. 6 is at most 38% and between Approach No. 5 and Approach No. 6 is at most 44%. Unlike the analysis conducted at 100 days, at 75 years, tensile stresses are observed for the decks used in these 4 projects when Approaches No. 4 – 6 are utilized, however they do not exceed the cracking strength. In all cases, Approach No. 6 predicts slightly lower tensile stresses when compared to other approaches. For the stresses along the cross-section of the decks used in these projects, the % difference between Approach No. 4 and Approach No. 6 is at most 389% and between Approach

No. 5 and Approach No. 6 is at most 404%. The higher % differences when compared to cross-sectional stresses for the beams is explain since the stresses along the deck cross-section for B05 project switch sign from positive to negative, which lead to division of change in stress to small number, and this results in very high numbers in terms of % differences.



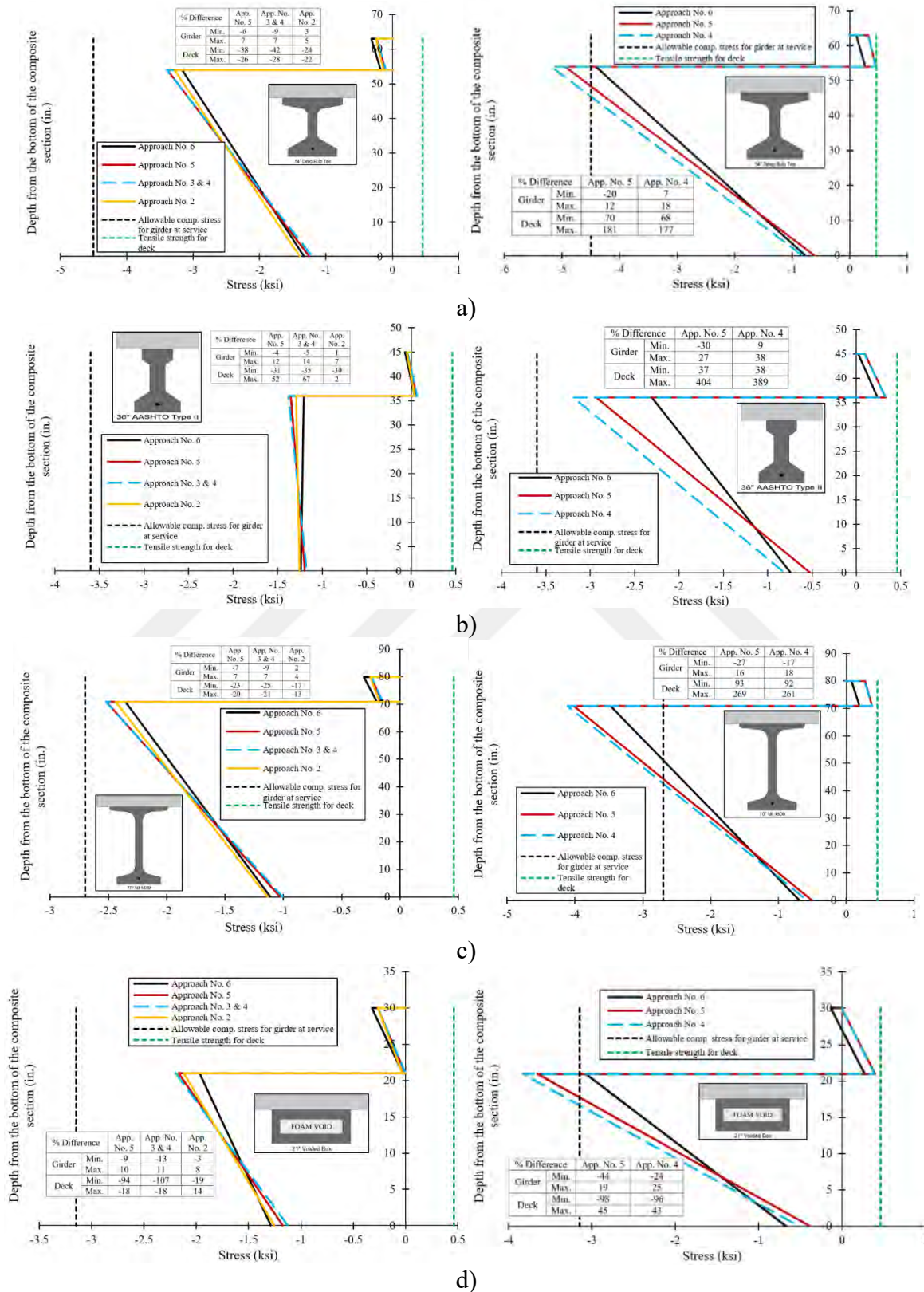


Figure 8.19. Comparison of cross-sectional stress analysis at 100 days (left) and 75 years (right) for: a) S11 project, b) B05 project, c) M57 project, and d) Shepherd Road-2 project

8.6.5. Impact of Method to Generate Time Steps within Approach No. 6

As described in section 3.5.6. Approach No. 6 produces time steps using a stepwise function in which more time steps are used at the beginning of an event and fewer at later times. Another approach is that proposed by Gilbert and Ranzi (2010) who suggest the use of a geometric series for the generation of the number of time steps. The impact of the method for generating the number of time steps used for a given loading event was evaluated by running Approach No. 6 one time using the stepwise function described in Table 3.3, and another time using the geometric series proposed by Gilbert and Ranzi (2010) using 1, 5, and 10 time substeps for every major time step. The results are shown in Figure 8.20 for S10-2 and S11 projects. The figures shown on the right hand side show the % differences between using a logarithmic time substep generation approach or using a geometric series based on a certain number of time substeps. Predictions based on the step wise function and the geometric series differ no more than 21% and 100% for S10-2 and S11 projects, respectively. The proposed prediction methodology allows the user to select either method for the generation of time steps for a given event.

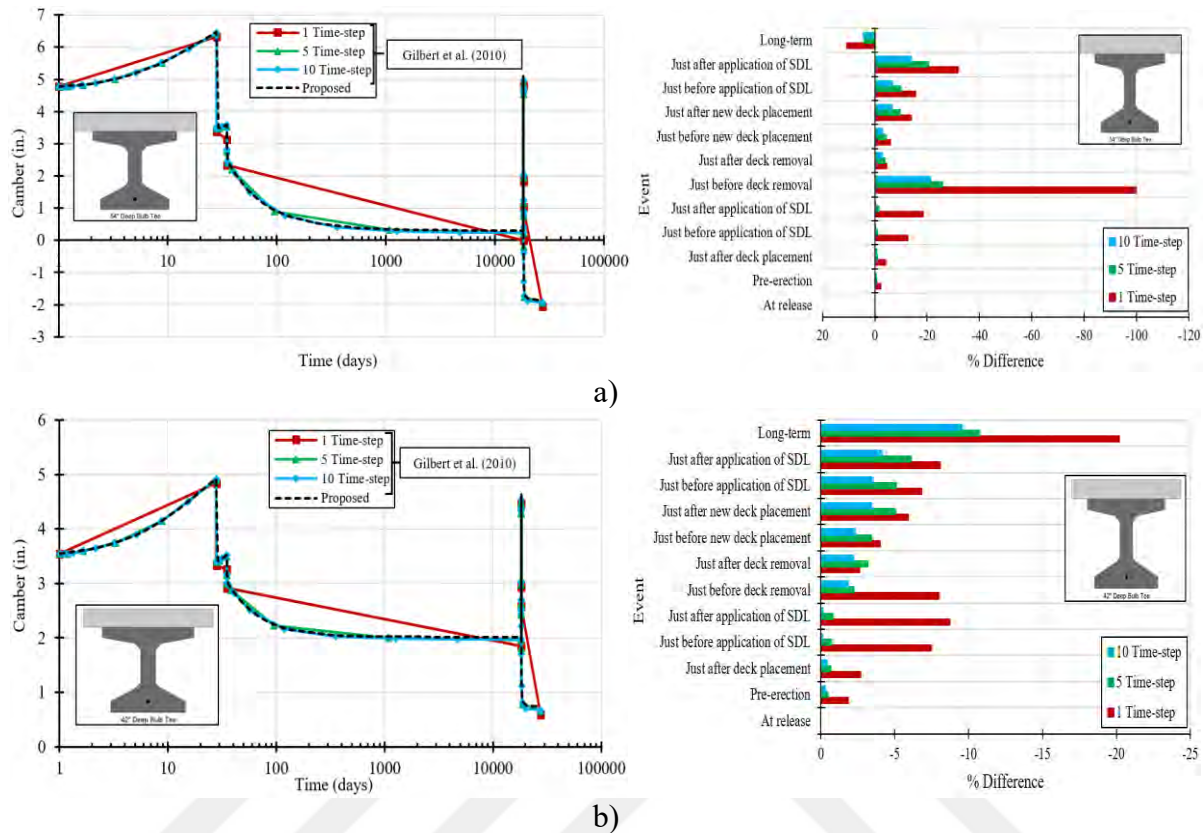


Figure 8.20. Impact of method to generate the number of time steps on full camber and displacement history for: a) S11 project, b) S10-2 project

8.6.6. Influence of Number of Beam Sections Specified in Approach No. 6

When using the MATLAB based algorithm (Approach No. 6) the user needs to specify the number of beam sections. This decision influences the prediction of camber, which is obtained by numerically integrating the curvature diagram along the beam span based on the selected number of sections. It also influences the smoothness of the deflected shape of the beam for a given time step. To obtain a realistic deflected shape, the user needs to specify enough sections so that the algorithm can produce a smooth deflected shape.

The sensitivity of camber predictions based on the approach used in Approach No. 6 to the number of beam sections selected was evaluated by varying the number of odd sections from 3 to 99 and the number of even sections from 2 to 100. The benchmark for conducting this evaluation is the camber predicted using 100 beam sections. The results of this evaluation are shown in Figure

8.21 for camber at release and pre-erection at midspan for the beams used in S11 project. Figure 8.21a suggest that the estimation of camber is not sensitive to the number of beam sections selected when this number is odd. For example, the difference in camber predictions between 3 and 100 beam sections is less than 0.5%. This shows the efficiency of the selected numerical integration technique when an odd number of beam sections are selected. Conversely, when the selected number of beam section is even, camber predictions become notably sensitive to the selected number of beam sections. For example, Figure 8.21b shows that the difference in camber prediction between 2 and 100 beam sections is greater than 35%. This difference reduces rapidly as the number of beam sections increases and becomes negligible when the number of sections exceeds 50. Therefore, it is recommended that at least 51 sections are selected when conducting a time-dependent analysis using the MATLAB based software developed based on Approach No. 6.

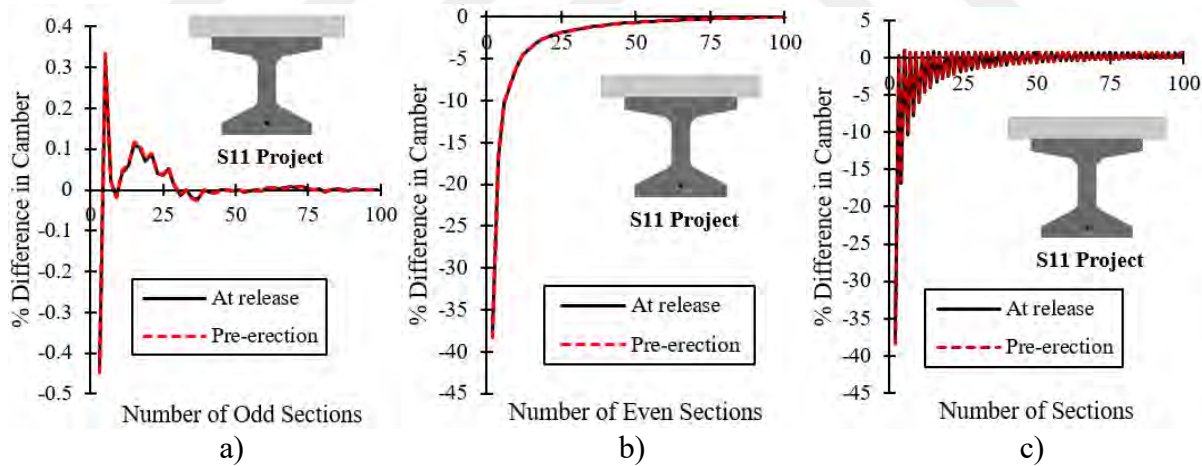


Figure 8.21. Sensitivity of camber predictions at midspan for the beams used in S11 project to the number of sections considered: a) number of odd sections, b) number of even sections, c) number of sections (odd or even)

8.7. Derivation of Time-Dependent Multiplier Method

Camber predictions obtained using the proposed prediction methodology were compared with those obtained using the time-dependent multiplier method. As explained in section 3.12, time-dependent multiplier curves were developed for each project based on results obtained from

the proposed prediction methodology (Approach No. 6). This was done by computing the ratio between long-term camber and camber at release (i.e. the multiplier) at various points in time up to 90 days for all prestressed concrete beams used in the projects considered as part of this thesis. These curves are shown in Figure 8.22. Then a logarithmic curve fit was conducted to develop a function that expresses the multiplier as a function of time (Eqn. 8-4).

If $t < 3$ days

$$\lambda = 1$$

otherwise $\lambda(t) = 0.17 \ln(t) + 0.85$ (λ : multiplier, and t : time in days) 8-4

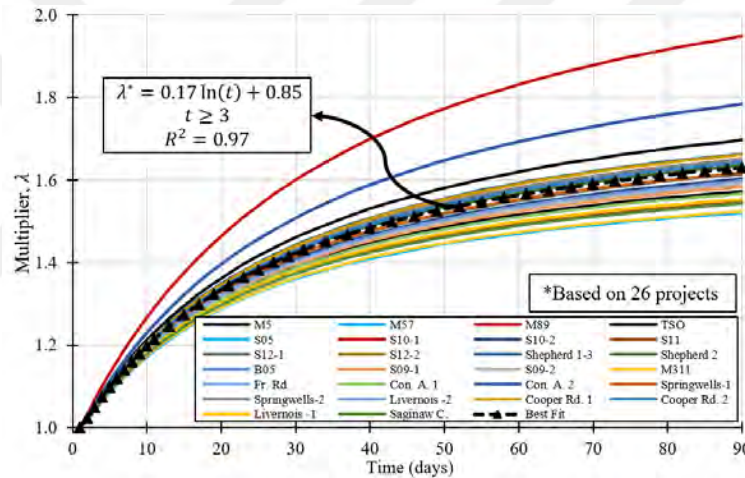


Figure 8.22. Variation of multipliers as a function of time (prediction curves for each project are based on specified properties)

In addition, a linear regression analysis was conducted to make the time-dependent multiplier equation shown in Eqn. 8-4 more generalized. As can be noticed in Figure 8.22, for some projects, predictions based on time-dependent multiplier equation deviates much from the predictions based on proposed prediction methodology (Approach No. 6). When the multipliers are developed using 26 different projects, M89 project provides the upper bound of multipliers whereas S05 projects provides the lower bound. When investigated these two, M57 and S05, projects which provides higher or lower multipliers, respectively, it is noticed that the projects M57 and S05 have the lowest and highest beam span lengths, L_G , and prestressing strand ratios,

ρ_{ps} , in the database considered in this thesis (Table 4.1). Therefore, a parametric analysis was conducted for some parameters such as beam length, L_G , prestressing steel ratio, ρ_{ps} , concrete compressive strength at release and at 28 days, f'_{ci} and f'_c , beam overhang length at precast facility, L_t , moment of inertia of the girder, I_G , centroid of girder with respect to bottom of girder, y_{bottom} , and girder height, h to determine the correlations of such parameters on pre-erection camber. Although sensitivity analysis conducted in CHAPTER 5 gives information about which parameters have the highest influence of pre-erection camber (Table 5.2), results presented in Figure 8.23a provides another perspective about the effect of such parameters on pre-erection camber by presenting the Pearson's correlation coefficients for investigated parameters. Based on Figure 8.23a, the first three parameters which provides the highest correlations for pre-erection camber are beam length, L_G , prestressing strand ratio, ρ_{ps} , and concrete compressive strength at release, f'_{ci} . Therefore, for the regression analysis, these three parameters as well as time parameter are considered. Based on the regression analysis using parameters t , L_G , ρ_{ps} and f'_{ci} , Eqn. 8-5 is obtained. With the inclusion of $r(t)$ factor shown in Eqn. 8-5, each project has its own time-dependent multiplier equation as illustrated in Figure 8.23b. To evaluate the inclusion of $r(t)$ factor, the project specific time-dependent multiplier curves (Eqn. 8-5) for two projects, M89 and S05, which provides the upper and lower bound multipliers, were compared with the Eqn. 8-4. Figure 8.23b suggests that the inclusion of $r(t)$ factor significantly improves the predictions and proposed time-dependent multiplier equation (Eqn. 8-5) provides reasonable predictions with the multipliers obtained proposed prediction methodology (Approach No. 6).

$$\lambda^*(t) = r(t)\lambda(t) \geq 1$$

$$\lambda(t) = 0.17 \ln(t) + 0.85$$

$$r(t) = 1.235 + 0.012 \frac{t}{t_o} - 0.039 \frac{L_G}{L_o} - 0.037 \frac{\rho_{ps}}{\rho_o} - 0.175 \frac{f'_{ci}}{f'_{ci_o}} \quad 8-5$$

$$\text{Where } t_o = 28 \text{ days; } L_o = 100 \text{ ft.; } \rho_{ps} = \frac{A_{ps}}{A_G}; \rho_o = 0.01 \text{ and } f'_{ci_o} = 6.5 \text{ ksi}$$

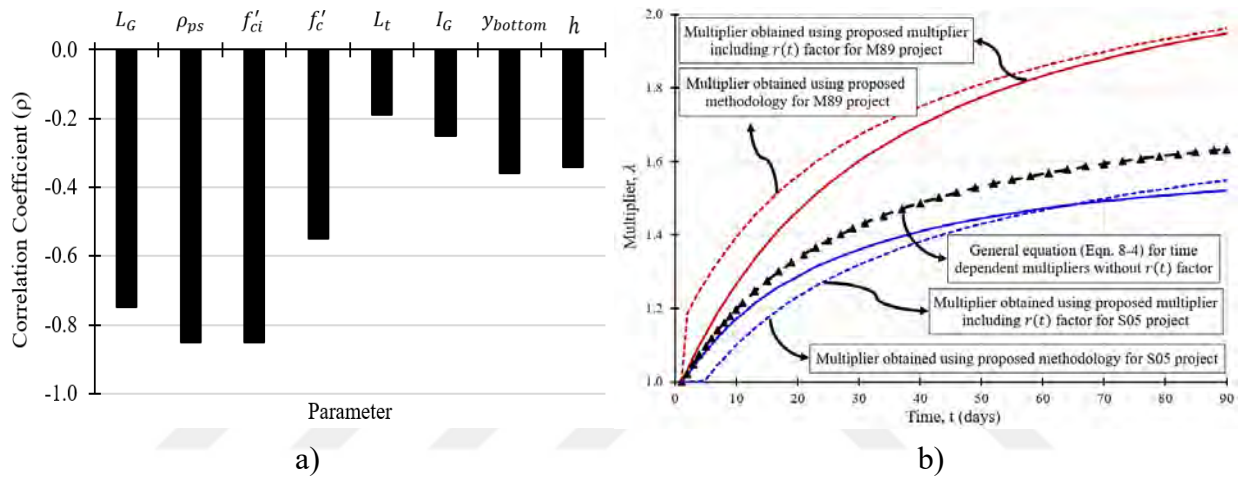


Figure 8.23. a) Influence of various parameters on pre-erection camber and b) Effect of $r(t)$ factor on the development of time-dependent multiplier equation

It should be noted that the time-dependent multiplier (Eqn. 8-5) is developed based on Approach No. 6 and is valid only for predicting pre-erection camber and cannot be used after the deck is cast. Such time-dependent multipliers may be useful and convenient during the preliminary design stage. This convenience should be used with caution as the proposed time-dependent multiplier values are empirically derived based on 26 different projects and may or may not be valid for all types of beams.

8.8. Comparison of Proposed Prediction Methodology and Time-Dependent Multiplier Method

Measured pre-erection camber was compared with predicted pre-erection camber based on the proposed prediction methodology (Approach No. 6), the proposed time-dependent multiplier method (Eqn. 8-5), MDOT's fixed multiplier method, and PCI's fixed multiplier method. The

comparison was done on a beam by beam basis using both specified and measured properties (Table 8.4). This comparison was conducted by calculating ratios of measured over predicted camber using each prediction methodology. Table 8.4 suggests that the proposed prediction methodology and the time-dependent multiplier method offer significantly better predictions than those obtained using PCI and MDOT multipliers. While the use of specified properties resulted overall in slightly more accurate predictions, the COV of was higher than when measured properties were used. Average ratios of measured over predicted camber when specified properties were used for the proposed prediction methodology and time-dependent multiplier method were 0.97 and 0.97; whereas those for the PCI and MDOT multiplier approach were 0.77 and 0.67, respectively. When measured properties were used, the average ratio of measured over predicted camber for the proposed prediction methodology and time-dependent multiplier method were 1.05 and 1.07; whereas those for the PCI and MDOT multiplier approach were 0.82 and 0.68, respectively.

Table 8.4. Comparison of several approaches for pre-erection camber

Statistical Indicators	Pre-erection Camber							
	Δ_m/Δ_{pmp_m}	Δ_m/Δ_{pmp_s}	Δ_m/Δ_{p_m}	Δ_m/Δ_{p_s}	Δ_m/Δ_{PCI_m}	Δ_m/Δ_{PCI_s}	Δ_m/Δ_{MDOT_m}	Δ_m/Δ_{MDOT_s}
Min.	0.71	0.67	0.71	0.67	0.48	0.46	0.38	0.40
Max.	1.54	1.47	1.53	1.50	1.36	1.13	1.30	1.00
Avg.	1.07	0.97	1.05	0.97	0.82	0.77	0.68	0.67
St. Dev.	0.18	0.17	0.17	0.18	0.14	0.14	0.14	0.12
COV (%)	17	18	17	18	18	18	20	18
# of beams	200	208	200	208	200	208	200	208

Δ_m = measured camber; Δ_{pmp_s} = predicted camber based on proposed multipliers and specified properties; Δ_{pmp_m} = predicted camber based on proposed multipliers and measured properties; Δ_{p_s} = predicted camber based on analysis and specified properties; Δ_{p_m} = predicted camber based on analysis and measured properties; Δ_{MDOT_s} = predicted camber based on MDOT multipliers and specified properties; Δ_{MDOT_m} = predicted camber based on MDOT multipliers and measured properties; Δ_{PCI_s} = predicted camber based on PCI multipliers and specified properties; Δ_{PCI_m} = predicted camber based on PCI multipliers and measured properties.

The time-dependent multiplier method was also evaluated by calculating the ratio of measured over predicted camber using initial camber predictions based on the proposed

methodology as well as measured initial camber. Comparison was also conducted once using specified properties and another time using measured properties. The initial camber predictions were based on specified and measured properties. This evaluation was done with the purpose of determining the benefit of using measured camber data when available to improve pre-erection camber predictions. To make a fair comparison, the beams which have measured camber at release were considered only. Therefore, the number of beams involved in this analysis was reduced.

Table 8.5 suggests that when measured camber at release together with measured concrete properties is used, the time-dependent multiplier method results in more accurate predictions compared to prediction based on predicted camber at release using measured concrete properties. However, the COV is larger indicating a lower degree of consistency in predictions when measured camber at release is used. On the other hand, when measured camber at release together with specified concrete properties is used, the time-dependent multiplier method do not results in more accurate predictions compared to prediction based on predicted camber at release using specified concrete properties. In addition, the COV is larger indicating a lower degree of consistency in predictions when measured camber at release is used.

Table 8.5. Comparison of proposed multiplier with different initial camber values

Statistical Indicators	Pre-erection Camber			
	Δ_m/Δ_{pmp_mp}	Δ_m/Δ_{pmp_sp}	Δ_m/Δ_{pmp_mm}	Δ_m/Δ_{pmp_sm}
Min.	0.72	0.68	0.66	0.64
Max.	1.54	1.20	1.58	1.42
Avg.	1.07	0.95	0.99	0.94
St. Dev.	0.22	0.17	0.23	0.21
COV (%)	20	18	24	22
# of beams	78	80	78	80

Δ_m = measured camber; Δ_{pmp_s} = predicted camber based on proposed multipliers and specified properties and predicted camber at release; Δ_{pmp_m} = predicted camber based on proposed multipliers and measured properties and predicted camber at release; Δ_{pmp_mm} = predicted camber based on proposed multipliers and measured properties and measured camber at release; Δ_{pmp_sm} = predicted camber based on proposed multipliers and specified properties and measured camber at release;

Finally, the time-dependent multiplier method was evaluated by comparing measured camber for Tiffin Street Overpass and Cooper Road (Hinkle 2006) projects. Analysis was conducted using specified and measured concrete properties. When measured properties were used, the average of all measured concrete values was considered. The results are shown in Figure 8.24. For Tiffin Street Overpass project (Figure 8.24a), the proposed time-dependent multiplier method slightly underestimates the measured camber whether specified or measured concrete properties are used. Although, the time-dependent multiplier method predicts camber growth rather well in beam G2-7, however the accuracy in prediction reduces for the other beams. It should be noted that there are some differences in measured camber despite the fact the beams are identical. These differences are attributed to inherent variability and unintended variation from specifications. As stated earlier, one of the reasons for the variation of measured camber could be the influence of temperature gradients, which can be evaluated using the *MDOTCamber* program. For the Cooper Road-1 project (Hinkle 2006), time-dependent multiplier method provides reasonable results, especially when measured concrete properties are used (Figure 8.24b). The use of specified properties decreases the accuracy in this case. Similarly, for the Cooper Road-2 project (Hinkle 2006), while the time-dependent multiplier method together with measured concrete properties provides relatively good results, the use of specified properties results in decrease in accuracy. In general, although it is difficult to propose about which concrete properties provides better results, it is demonstrated that the proposed time-dependent multiplier method provides significantly better results than PCI and MDOT multiplier methods.

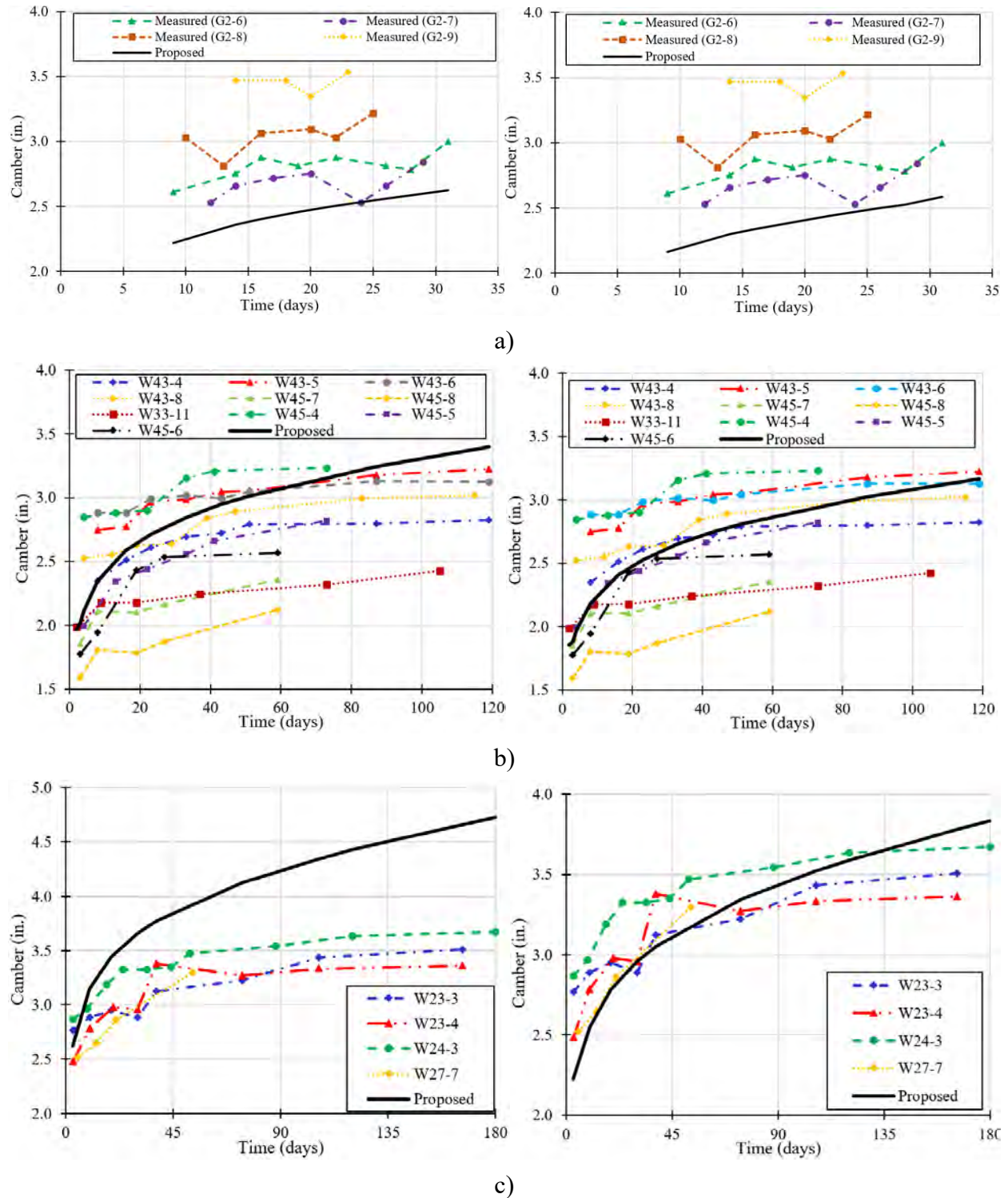


Figure 8.24. Camber growth prediction using proposed time-dependent multiplier method and specified (left) and measured (right) properties for: a) Tiffin Street Overpass Project, b) Cooper Road-1 Project (Hinkle 2006, Strand pattern I) and c) Cooper Road-2 Project (Hinkle 2006, Strand pattern II)

8.9. Summary and Conclusions

- 1) With respect to predictions for camber at release, the proposed prediction methodology resulted in more accurate and consistent predictions than the current MDOT method regardless of whether measured or specified properties are used. Although the use of specified properties provided slightly better average of measured over predicted camber ratio, the COV was higher than the results obtained using measured properties.
- 2) With respect to predictions for pre-erection camber the proposed prediction methodology resulted in more accurate and consistent predictions compared to the PCI multiplier method and MDOT multiplier method regardless of whether measured or specified properties are used. Although the use of specified properties provided slightly better average of measured over predicted camber ratio, the COV was higher than the results obtained using measured properties. Additionally, predictions based on the proposed Time-Dependent Multiplier Method and Incremental Time Step method (Nilson 1987) were also more accurate compared to the PCI Multiplier Method and the MDOT multiplier method.
- 3) Camber growth curves were developed for non-composite structure using various creep and shrinkage models together with specified and measured concrete properties. The results were compared with the periodically measured camber data. Due to large scatter in the measured data, a firm conclusion about which model provided the best prediction could not be made, however, the use of measured properties generally improved the predictions.
- 4) Camber growth curves were developed for composite prestressed concrete and steel structure using both body and commentary provisions of AASHTO LRFD 2020. The results were compared with the periodically measured camber data. For prestressed

concrete beams, commentary provision of AASHTO LRFD 2020 generally provided better results than the body provision whereas for the post-erection zone, overall, the body provision of AASHTO LRFD 2020 provided better match with the measured data. For the steel beams, the commentary provision of AASHTO LRFD 2020 predicted the service camber better than body provision.

- 5) Prestress loss histories were evaluated using 4 different cross-sectional types of beams which are bulb tee, AASHTO Type II, MI 1800 (Michigan based girder) and Voided-Box. It was concluded that girder creep causes the majority of prestress loss. Girder shrinkage is the second largest source of prestress loss.
- 6) Prestress losses predicted based on Approach No. 6 and those predicted using AASHTO LFRD Bridge Design Specifications (2020) were similar. The Incremental Time Step Method (Nilson 1987) and Approach No. 6 led to rather similar predictions of pre-erection camber.
- 7) Most pre-erection camber and full displacement history predictions were based on creep and shrinkage models provided in the body of AASHTO LFRD Bridge Design Specifications (2020), the AASHTO LRFD 2020 equation for predicting modulus at prestress release and 28 days, and ACI 209R-92 model with back-calculated parameters for predicting the variation of modulus with time. When compared to measured pre-erection camber values, this set of prediction models, inserted in the proposed prediction methodology, resulted in reasonably accurate pre-erection camber predictions. The proposed methodology allows the user to select various models for creep and shrinkage of beam and deck concrete, prediction of modulus at prestress release and 28 days, and its

variation with time. The user is encouraged to evaluate the use of different models to obtain a sense for the anticipated variation of predicted camber and displacements.

- 8) Differences between predicted pre-erection camber based on Approaches No. 1 – 5 and Approach No. 6 did not exceed 4%. Differences between predicted service camber without deck replacement based on Approaches No. 2 – 5 and Approach No. 6 did not exceed 947%. Differences between predicted service camber with deck replacement based on Approaches No. 3 – 5 and Approach No. 6 did not exceed 5022%. Since the service camber at 75 years was close to 0 for some projects, the division of change in camber to the net camber caused these big % difference results. In most of the cases, the max absolute difference between predictions are at most around 1 inch.
- 9) The prestressing strand stress histories were compared using various approaches. It was observed that the stress in the prestressing strands did not vary much when different approaches were used.
- 10) The stress histories for the top and bottom fibers of prestressed concrete beam projects and the decks used in these projects were compared using various approaches and allowable limits based on AASHTO LRFD 2020. It was concluded that there was not a significant difference between the results when different approaches were employed.
- 11) The impact of method to generate time steps for conducting time-dependent analysis was investigated using proposed prediction methodology. The proposed method and methods provided by Gilbert and Ranzi (2010) were compared to each other, and it was concluded that the results were almost identical between proposed and Gilbert and Ranzi (2020) time step generation methods after 5 time steps were used per interval.

- 12) The proposed prediction methodology (Approach No. 6) requires that the user specifies the number of beam sections to be used in the time dependent analysis. It is recommended that this number is at least 51 to obtain accurate results.
- 13) A new regression based Time-dependent Multiplier Method was derived for predicting pre-erection camber. This method should be used for preliminary design purposes and, the proposed prediction methodology (Approach No. 6) should be used for final design. The proposed time-dependent multiplier method led to more accurate predictions of pre-erection camber compared to the PCI multiplier method and MDOT multiplier method.

CHAPTER 9: Summary and Conclusions

A methodology for predicting camber and displacements in prestressed concrete and steel girder bridges was presented. For prestressed concrete beams the prediction methodology can be used to predict camber and displacements at any point in time during the life of the beam. For steel beams, the prediction methodology can be used to predict camber and displacements in the composite bridge system. The proposed methodology was based on 6 approaches each of which represents a theoretical improvement in prediction accuracy compared to the previous approach. The last approach (i.e. Approach No. 6) is presented as an algorithm written in Matlab that features a graphical user interface. In addition to the proposed prediction methodology, two alternative methods for predicting pre-erection camber were evaluated. These include the Incremental Time Step method (Nilson 1987) and the Time-dependent Multiplier Method. The Time-dependent Multiplier Method was derived from the proposed prediction methodology using curve-fitting techniques. The following conclusions are drawn:

9.1. Chapter 3 - General:

- 1) The proposed prediction methodology can be used to predict camber and displacements at any point in time during the life of the bridge including cases in which the deck is replaced. The proposed methodology may be used for prestressed concrete and steel girder bridges.
- 2) The proposed prediction methodology considers the effects of concrete creep, shrinkage, and ageing, as well as steel relaxation. The effects of differential shrinkage and differential creep as well as shrinkage induced creep are also considered. The proposed prediction methodology considers the effects of temperature gradients thus providing bracketed predictions for camber and displacements at any point in time.

- 3) Camber and displacement predictions obtained from the proposed prediction methodology are blind predictions, in the sense that no calibration is conducted to match measured camber values based on Michigan specific conditions, and all predictions are based on principles of engineering mechanics when internal force redistribution is considered. Empirical models are used to predict modulus of elasticity, creep and shrinkage of concrete, relaxation of prestressing strands, and expected concrete compressive strength.

9.2. Chapter 4 – Relationships between Specified and Measured Properties

- 4) Overstrength factors 1.20 and 1.20 at prestress transfer and 28 days were proposed for Michigan concrete based on 299 and 234 data, respectively.
- 5) Based on 9 tests, the modulus of elasticity predictions based on the fib MC 2010 model provided the best match with the measured values. Comparison based on AASHTO LRFD 2020 model provided the second best predictions. Overall, the use of measured properties improved the predictions.
- 6) It was shown that ACI 209R-92 model with back-calculated parameters predicted the modulus of elasticity with time with reasonable accuracy. The use of measured properties improved the predictions.
- 7) It was concluded that the measured values of unit weight of concrete, prestressing force, and beam length do not show significant variation from the specified ones. Therefore, they may be eliminated as a source of uncertainty in the camber predictions.
- 8) A couple of solar radiation studies were conducted to explore the temperature induced camber. It was concluded that the camber could vary as much as 0.22 in. due to temperature gradients.

9.3. Chapter 5 - Factors that Affect Prestressed Concrete Beam Camber

- 9) While it is determined that the unit weight of concrete, w , the magnitude of the prestressing force at jacking, $P_{jacking}$, and beam length, L , all have a significant influence on camber at release and pre-erection beam camber, it is determined that these parameters do not vary significantly from specified values and therefore do not represent a significant source of uncertainty.
- 10) Modulus of elasticity at release, E_{ci} , had a proportional influence on camber at release and pre-erection beam camber.
- 11) Similarly, beam compressive strength at release, f'_{ci} , also had a close to proportional influence on camber at release and pre-erection camber, although this influence was quantified through the use of compressive strength dependent equation for modulus of elasticity.
- 12) Transfer length, $L_{transfer}$, debonded length, $L_{debonded}$, support conditions during storage, $L_{overhang}$, and location of harping point, $L_{harping}$, influence camber at release and pre-erection camber at a degree that in some cases is worth considering.
- 13) The selection of the creep and shrinkage model has a marked influence on the prediction of pre-erection camber development.
- 14) The selection of modulus of elasticity model has a marked influence on the prediction of camber at release and pre-erection camber.
- 15) The selection of model for capturing the variation of modulus of elasticity with time was inconsequential. The % difference in pre-erection camber predictions is less than 0.1%.

- 16) The inclusion of overstrength factors had a marked effect on pre-erection camber predictions with differences between the two models being up to 10%. The inclusion of overstrength factors results in lower camber predictions.
- 17) The time when camber at release is measured appears to be an important parameter since marked differences were found between predicted camber at release and predicted camber during the first 10 days. The proposed prediction methodology (Approach No. 6) provides the user the flexibility of accounting for the influence of all the abovementioned factors.

9.4. Chapter 6 - Factors that Affect Camber and Displacements in the Composite System

- 18) Factors that led to a single digit average % change in net camber and displacement due to the induced 10% change include: beam overhang length at precast facility (i.e. storage conditions), transfer length, debonded length, deck modulus, and beam overhang length at the bridge site.
- 19) Factors that led to a double digit average % change in net camber and displacement due to the induced 10% change include: beam concrete compressive strength at release and 28 days, beam concrete unit weight, beam concrete modulus of elasticity at release and 28 days, location of harping point, beam spacing, compressive strength of deck at 28 days, and unit weight of deck.
- 20) Factors that led to a triple digit average % change in net camber and displacement due to the induced 10% change include: beam length, and prestressing force.
- 21) The influence of the selected creep and shrinkage model on the full displacement history of a prestressed concrete beam bridge was investigated and it was concluded that this selection has a marked influence on the beam displacement history. Some models result in

rather similar displacements after 75 years despite initial differences in pre-erection camber and net displacements after deck placement.

- 22) The selection of modulus of elasticity model has a notable influence on the prediction of service camber.
- 23) The selection of model for capturing the variation of modulus of elasticity with time was inconsequential. The % difference in pre-erection camber predictions is less than 0.5%.
- 24) The inclusion of overstrength factors had a marked effect on service camber predictions with differences between the two models being up to 160%. The inclusion of overstrength factors results in lower camber predictions.
- 25) Influence of deck placement time on the full beam displacement history was investigated and it was concluded that while pre-erection camber is highly influenced by it, camber and displacements after 75 years were rather similar.
- 26) The influence of temperature gradients was rather uniform throughout the displacement history of the beam with positive temperature gradients having a higher influence on camber and displacements compared to negative temperature gradients.

9.5. Chapter 7 - Factors that Affect Beam Rebound and Behavior after Deck Replacement

- 27) Deck replacement time had no influence on the magnitude of beam rebound and net camber after deck removal provided that the deck is replaced at least after 40 years. Deck replacement time had a minor influence on the net camber before new deck placement with greater deck replacement times resulting in slightly lower net cambers before new deck placement.
- 28) It was found that while steel beams are able to rebound to their original position after the deck is removed, the rebound in prestressed concrete beams varies from 47%-92%.

29) The influence of solution method (Approach No. 5 vs. Approach No. 6) on beam rebound, net camber after deck removal, and net camber before new deck placement was investigated. It was determined that Approach No. 6 leads to smaller rebounds compared to Approach No. 5. Additionally, predictions based on Approach No. 5 showed that there is no change between net camber after deck removal, and net camber before new deck placement whereas Approach No. 6 suggests that there is a slight camber growth.

9.6. Chapter 8 - Evaluation of Various Design Procedures

- 30) With respect to predictions for camber at release, the proposed prediction methodology resulted in more accurate and consistent predictions than the current MDOT method regardless of whether measured or specified properties are used. Although the use of specified properties provided slightly better average of measured over predicted camber ratio, the COV was higher than the results obtained using measured properties.
- 31) With respect to predictions for camber at release, the proposed prediction methodology resulted in more accurate and consistent predictions than the current MDOT method regardless of whether measured or specified properties are used. Although the use of specified properties provided slightly better average of measured over predicted camber ratio, the COV was higher than the results obtained using measured properties.
- 32) With respect to predictions for pre-erection camber the proposed prediction methodology resulted in more accurate and consistent predictions compared to the PCI multiplier method and MDOT multiplier method regardless of whether measured or specified properties are used. Although the use of specified properties provided slightly better average of measured over predicted camber ratio, the COV was higher than the results obtained using measured properties. Additionally, predictions based on the proposed Time-Dependent Multiplier

Method and Incremental Time Step method (Nilson 1987) were also more accurate compared to the PCI Multiplier Method and the MDOT multiplier method.

- 33) Camber growth curves were developed for non-composite structure using various creep and shrinkage models together with specified and measured concrete properties. The results were compared with the periodically measured camber data. Due to large scatter in the measured data, a firm conclusion about which model provided the best prediction could not be made, however, the use of measured properties generally improved the predictions.
- 34) Camber growth curves were developed for composite prestressed concrete and steel structure using both body and commentary provisions of AASHTO LRFD 2020. The results were compared with the periodically measured camber data. For prestressed concrete beams, commentary provision of AASHTO LRFD 2020 generally provided better results than the body provision whereas for the post-erection zone, overall, the body provision of AASHTO LRFD 2020 provided better match with the measured data. For the steel beams, the commentary provision of AASHTO LRFD 2020 predicted the service camber better than body provision.
- 35) Prestress loss histories were evaluated using 4 different cross-sectional types of beams which are bulb tee, AASHTO Type II, MI 1800 (Michigan based girder) and Voided-Box. It was concluded that girder creep causes the majority of prestress loss. Girder shrinkage is the second largest source of prestress loss.
- 36) Prestress losses predicted based on Approach No. 6 and those predicted using AASHTO LRFD Bridge Design Specifications (2020) were similar. The Incremental Time Step Method (Nilson 1987) and Approach No. 6 led to rather similar predictions of pre-erection camber.

- 37) Most pre-erection camber and full displacement history predictions were based on creep and shrinkage models provided in the body of AASHTO LRFD Bridge Design Specifications (2020), the AASHTO LRFD 2020 equation for predicting modulus at prestress release and 28 days, and ACI 209R-92 model with back-calculated parameters for predicting the variation of modulus with time. When compared to measured pre-erection camber values, this set of prediction models, inserted in the proposed prediction methodology, resulted in reasonably accurate pre-erection camber predictions. The proposed methodology allows the user to select various models for creep and shrinkage of beam and deck concrete, prediction of modulus at prestress release and 28 days, and its variation with time. The user is encouraged to evaluate the use of different models to obtain a sense for the anticipated variation of predicted camber and displacements.
- 38) Differences between predicted pre-erection camber based on Approaches No. 1 – 5 and Approach No. 6 did not exceed 4%. Differences between predicted service camber without deck replacement based on Approaches No. 2 – 5 and Approach No. 6 did not exceed 947%. Differences between predicted service camber with deck replacement based on Approaches No. 3 – 5 and Approach No. 6 did not exceed 5022%. Since the service camber at 75 years was close to 0 for some projects, the division of change in camber to the net camber caused these big % difference results. In most of the cases, the max absolute difference between predictions are at most around 1 inch.
- 39) The prestressing strand stress histories were compared using various approaches. It was observed that the stress in the prestressing strands did not vary much when different approaches were used.

- 40) The stress histories for the top and bottom fibers of prestressed concrete beam projects and the decks used in these projects were compared using various approaches and allowable limits based on AASHTO LRFD 2020. It was concluded that there was not a significant difference between the results when different approaches were employed.
- 41) The impact of method to generate time steps for conducting time-dependent analysis was investigated using proposed prediction methodology. The proposed method and methods provided by Gilbert and Ranzi (2010) were compared to each other, and it was concluded that the results were almost identical between proposed and Gilbert and Ranzi (2020) time step generation methods after 5 time steps were used per interval.
- 42) The proposed prediction methodology (Approach No. 6) requires that the user specifies the number of beam sections to be used in the time dependent analysis. It is recommended that this number is at least 51 to obtain accurate results.
- 43) A new regression based Time-dependent Multiplier Method was derived for predicting pre-erection camber. This method should be used for preliminary design purposes and, the proposed prediction methodology (Approach No. 6) should be used for final design. The proposed time-dependent multiplier method led to more accurate predictions of pre-erection camber compared to the PCI multiplier method and MDOT multiplier method.



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APPENDIX A: Previous Studies

A.1. Buettner and Libby (1979)

Buettner and Libby (1979) explored several serviceability issues caused by inaccurate predictions of camber in prestressed concrete girders. The study documented the variation in the predicted and measured camber for 37 short span (less than 50 feet) bridge girders produced in Fairfax County, VA. The study concluded that there was consistent variability in the measured camber and the computational method used resulted in over predictions of camber. The errors in prediction were attributed to (1) inaccurate assumptions for the concrete compressive strength and modulus of elasticity, (2) using uncracked concrete properties when the section may have actually cracked, (3) inaccuracies in the assumed magnitude of the prestressing force, (4) shipping and erection times being inconsistent and/or inaccurate, and (5) a lack of consideration of the influence of temperature gradients. It was concluded that: (1) camber at release should be accurately recorded for all projects, (2) predicted camber should be noted in the shop drawings, and (3) the difference between the predicted and measured camber shall not exceed $L/1200$. This research was the first to document the growing issues of inaccurate camber prediction and correctly hypothesized many probable causes for such inaccuracies.

A.2. Tadros et al. (1985)

Tadros et al. (1985) investigated methods for how to more accurately predict long-term camber by considering the influence of time-dependent effects in prestressed concrete members. The goal of this study was to improve the predictions based on the PCI multiplier method proposed by Martin (1977) by considering the effects of higher strength concrete, relative humidity on creep and shrinkage, presence of non-prestressed steel, and concrete cracking. The developed method was a more elaborate multiplier method, which produced similar results in typical bridge girders under average environmental conditions with those obtained based on Martin's method. Despite

this revision to the original multiplier method, the multipliers used in the PCI Bridge Design Manual (Precast/Prestressed Concrete Institute 2011) are unchanged and are still based on the work of Martin (1977).

A.3. Kelly et al. (1987)

Kelly, Bradberry, and Breen (1987) investigated camber growth in eight (8) – 127 ft long AASHTO Type IV high strength concrete bridge girders with low-relaxation steel tendons. The girders were monitored over a period of time, starting at the time of girder production and ending a year after entering service. The measurements during this period of time included concrete surface strains, prestressing strand strains, and deflections at quarter and mid-span. Internal beam temperature was also monitored and recorded. This set of data was used in evaluating various time-dependent methods used to calculate deflections. The study concluded that the rate at which concrete strength varies with time, concrete creep, relative humidity, age and strength of concrete at release, construction schedule, support conditions during storage are all factors, which influence time-dependent camber. The study also concluded that the introduction of regional practices into the PCI multiplier method increased the accuracy of predicting time-dependent camber.

A.4. Brown (1998)

Brown (1998) examined the methods for predicting camber growth used by the Idaho Department of Transportation (IDOT) for prestressed concrete girders at that time. Camber measurements were collected from four different prestressed concrete girder manufacturers and compared to camber predictions computed by then-current IDOT design procedures. It was found that the prediction method tended to underestimate camber at the time of prestress release. Primary conclusions from the study included the following: (1) the primary contributor to prestress losses at release is elastic shortening, while losses due to steel relaxation are minimal and can be

neglected, (2) the incremental time-steps method provided an accurate prediction of camber after calibration of creep and shrinkage coefficients, (3) a modified PCI multiplier method was developed to include local concrete material properties and a regionally appropriate construction timeline, and (4) relative humidity was not observed to significantly affect camber growth. It was stressed that the recommended camber prediction methods are based on estimates of the modulus of elasticity, ultimate creep coefficient, and ultimate shrinkage strain and should be validated by a future material testing program. Various methods to control camber growth in girders were also explored.

A.5. Yazdani et al. (1999)

Yazdani et al. (1999) monitored the camber growth in several AASHTO girders as part of a study for the Florida Department of Transportation (FDOT) to identify some of the reasons for the problems encountered during construction as a result of inaccurate camber predictions. It was determined that microcracks and vibrations affect girder stiffness and consequently camber predictions. Measured camber was always greater than predicted camber. It was also determined that the rate of camber growth was higher during the first month of storage and that camber in shorter girders was found to increase quicker than camber in longer girders. The average variation between predicted and measured camber ranged from 3-11% with maximum variations as high as 20%.

A.6. Wyffels et al. (2000)

Wyffels et al. (2000) investigated the influence of pre-release cracking due to formwork removal on prestressed concrete beam behavior for the Minnesota Department of Transportation (MnDOT) and concluded that: (1) compressive strength at the bottom of a section may be reduced as a result of cracks closing if the majority of pre-release cracking occurs above the neutral axis of

the girder, and (2) bridge girders that have widespread pre-release cracking exhibit a reduced camber effect. It was noted that experimental testing is needed to validate these conclusions because despite evidence from the analytical models, some researchers still argue that pre-release cracking in precast, prestressed concrete bridge girders produces negligible effects due to the closing of the cracks upon prestress release and the corresponding autogenous healing which may occur thereafter.

A.7. Jáuregui et al. (2002)

Jáuregui et al. (2002) explored the use of digital close-range terrestrial photogrammetry (DCRTP) to estimate deflections in bridges. Two bridge structures were analyzed using DCRTP; a new bridge constructed with prestressed concrete, and a 64-year-old non-composite steel girder bridge. DCRTP measurements taken from the new bridge were compared with field-measurements during pre-erection, deck casting, and at service. The DCRTP measurements were found to compare well to field measured data. Also, live load deflections in the steel bridge were estimated using DCRTP and were found to be precise enough to capture the very small deflections produced due to live loads. It was recommended that DCRTP could be used to accurately examine deflections and structural behavior in lieu of specialized instrumentation.

A.8. Stallings, Barnes, and Eskildsen (2003)

Stallings, Barnes and Eskildsen (2003) conducted a study for the Alabama Department of Transportation (ALDOT), which focused on evaluating various camber and prestress loss prediction methods for bridge girders constructed with high-performance concrete (HPC). Five BT-54 girders were instrumented and monitored. Mid-span camber, concrete strain, and internal temperature were measured and recorded throughout the duration of the project. Creep, shrinkage and modulus testing was conducted using HPC samples from the girders to determine such

properties. The study concluded that: (1) By using actual material properties of the girder, both incremental and approximate time-step methods can produce accurate predictions of camber, (2) The PCI multiplier method significantly overestimated camber when compared to measured pre-erection camber, (3) strains in the concrete at the prestressing strand level was within 20 percent of predicted values for times up to 300 days, (4) accurate predictions of time-dependent prestress loss can be obtained using the HPC material parameters.

A.9. Waldron (2004)

Waldron investigated long-term prestress losses in pretensioned high performance concrete girders and monitored changes in strains and prestresses losses for none high performance concrete (HPC) girders. It was concluded that Shams and Kahn (2010) model resulted in the best predictions of strain. For the normal weight HPC girders, the considered creep and shrinkage models underestimated the measured strains at early ages and over-estimated the measured strains at later ages, and the B3 model was the best-predictor of the measured strains. The PCI-BDM model was the most consistent model across all of the instrumented girders (Waldron 2004). Several methods for estimating prestress losses were also investigated and the PCI-BDM and NCHRP 496 methods predicted the total losses more accurately than the methods provided in the AASHTO Specifications. The newer methods over-predicted the total losses of the HPLWC girders by no more than 8 ksi, and although they under-predicted the total losses of the normal weight HPC girders, they did so by less than 5 ksi (Waldron 2004).

A.10. Cook and Bloomquist (2005)

Cook and Bloomquist (2005) compared camber measurements of prestressed girders to numerical predictions. Funded by the Florida Department of Transportation (FDOT), part of the study monitored time-dependent deformations and surface temperatures of 13 precast prestressed

concrete bridge girders. Data collected from the 13 girders was compared to predicted camber using the software package PSBEAM. To account for the effects of temperature gradients on field-measured camber, a curvature-based model was used, which provided consistent comparisons between measurements regardless of ambient conditions. The research concluded that: (1) the observed camber increase with time for the field-monitored girders was significantly less than predicted by design software, (2) future work is needed to experimentally determine the creep and shrinkage properties of typical FDOT concretes, (3) the influence of thermal gradients on camber must be accounted for in field measurements, (4) storage conditions (namely, the height between the bottom of the girder and the ground) seem to affect the development of time-dependent concrete properties and, therefore, can affect the time-dependent development of camber, and (5) consistent differences were documented between camber measurements taken at prestress release and measurements taken shortly thereafter when girders were relocated to storage. Also, FDOT sponsored a follow-up study performed by Tia, Liu, and Brown (2005) to examine modulus of elasticity, creep, and shrinkage behavior for various FDOT concrete mixes.

A.11. Barr et al. (2005)

Barr et al. (2005) examined five precast, prestressed concrete girders throughout fabrication and service to determine the influence of elevated curing temperatures on prestress losses. The study was funded by the Washington Department of Transportation (WSDOT) and the data collected was also used to produce and validate a temperature correction procedure based on curvature. The study concluded that: (1) based on the observed thermal behavior of the girders, a curvature-based approach was developed and proved to accurately predict deformations and stresses due to thermal response, and (2) mid-span camber decreases significantly from elevated

curing temperatures observed in precast, prestressed concrete due to significant loss in prestressing force.

A.12. Hinkle (2006)

Hinkle (2006) used data collected from twenty-seven high-strength prestressed bridge girders to determine the most accurate method to predict time-dependent camber. The study, conducted in South Carolina, used 79 in. Bulb Tee beams with a design compressive strength of 9,000 psi. The girders were periodically measured to determine camber growth over time. An incremental time-step method was employed using changes in beam curvature to predict camber and compare it with measured values. The study considered the effect of season of casting as well as the effects of solar radiation on camber growth. It was concluded that: (1) camber in girders could vary up to 0.5 inches during the day due to solar exposure, (2) the PCI multiplier method tended to overestimate camber by 48 percent at an age of 60 days, (3) the revised PCI multiplier method (proposed by Tadros et al. 1985) tended to overestimate camber by 21 percent at an age of 60 days, and (4) for all 27 beams studied, the incremental time-step method using the estimated creep, and shrinkage, as well as modulus of elasticity calculated based on the model developed by Shams and Kahn (2000), resulted in camber predictions that best matched the measured camber values.

A.13. Rosa et al. (2007)

Rosa et al. (2007) collected short term camber data for 146 girders and long term camber data for 91 girders as part of a study funded by WSDOT. Camber was measured at various girder ages. Data on compressive strength, modulus, creep and shrinkage behavior of girders were also recorded. These data were then used to calibrate constitutive models for use in a prediction framework. The calibrated material models were used in an incremental time-step method and

resulted in improved camber predictions. Recommendations regarding concrete strength, stiffness, and creep and shrinkage behavior were presented. Additionally, it was concluded that the effect of lifting and re-seating a girder tended to increase the measured camber by 0.15 in., and that girders stored on oak blocks tended to behave as if they were approximately 50 percent stiffer than those seated on elastomeric bearings (attributed to the partial restraint provided by oak blocks).

A.14. Jayaseelan and Russell (2007)

Jayaseelan and Russell (2007) studied prestress losses and camber in an AASHTO Type IV girder detailed to typical Oklahoma Department of Transportation (OKDOT) specifications and arrived at the following conclusions: (1) expected camber was reduced by 35% by adding two top prestressing strands and 70% by adding 4 strands, (2) long-term camber decreased by approximately 17% due to the addition of five No. 9 mild steel bars in the longitudinal direction, although this did not significantly affect long-term losses, (3) decreasing the creep coefficient by 20% yields a 6.8% decrease in long-term camber, and (4) reducing the modulus of elasticity by 20% yields 6% increase in long-term prestress loss and a 12% increase in long-term camber.

A.15. Omar et al. (2008)

Omar et al. (2007) conducted a study on camber prediction for prestressed concrete girders in Malaysia where “camber” is referred to as “pre-camber”. Similar to other researchers, Omar recommended the use of actual material properties in terms of concrete compressive strength and stiffness for computation of pre-camber. It was determined that the time-dependent change in pre-camber after girder fabrication could be modeled by a proposed equation similar to the PCI multiplier method. The proposed equation takes into account creep effects and assumes an average prestressing force. Through the proposed method, it was found that the accuracy of pre-camber predictions could be improved significantly for time periods up to 15 days from girder fabrication.

A.16. Barr and Angomas (2010)

Barr and Angomas (2010) revisited a previous study conducted by Barr et al. (2005) and offered a revised analytical procedure. The new procedure provided a better match between predicted and observed behavior in the field in the precast, prestressed concrete girders used in the study. It was found that: (1) computed reduction in camber due to high curing temperatures was 33% rather than 40% as previously calculated, (2) a reduction in strand stress and non-uniform temperature gradients at estimated time of bonding are produced from elevated curing temperatures, which cause a change in camber, (3) by employing an incremental time-step method and using material properties as suggested by Tadros et al. (2003) in NCHRP Report 496, camber was predicted to within 10% of measured long-term camber, and (4) camber was observed to be 22% lower than that obtained using the PCI multiplier method (Martin 1977), and 27% higher than that obtained using the modified multiplier method developed by Tadros et al. (1985).

A.17. Lee (2010)

Lee (2010) investigated thermal effects in prestressed concrete girders both experimentally and analytically. Using a segment of a BT-63 prestressed concrete girder, Lee measured environmental conditions and created finite element models to calculate vertical and lateral displacements. Both a two-dimensional heat-transfer model and a three-dimensional solid model were used in the analysis. After validating the analytical model with experimental data, four PCI girder shapes were selected, and maximum vertical and lateral deflections due to thermal effects were predicted with the model. It was found that the deeper and wider sections of Type-V and BT-63 girders exhibited the largest vertical and transverse temperature differentials. The research suggested that more experimental and analytical investigations be performed on prestressed concrete girders from the construction stages until service load conditions.

A.18. Tadros et al. (2011)

Tadros et al. (2011) reviewed the most-current camber prediction methods for precast, prestressed concrete girders and also discussed the variability in camber and best practices for accommodating this variability in design. The following were concluded: (1) when detailing bridges, designers should consider camber variations of up to 50% from the predicted design values, (2) all bridges should be designed with a minimum girder haunch of 2.5 in., (3) shear reinforcement should be detailed to accommodate camber variability by keeping protruding bars vertical prior to erection and bending on-site to final elevations, (4) girder seats should be finalized near the time of girder installation to accommodate variable elevations, (5) contractor pay items based on concrete volume should be avoided and instead, the contractor should account for girder variability in their initial bid, and (6) designers should accommodate local material properties and storage and construction practices during design, if practical.

A.19. French and O'Neill (2012)

French and O'Neill (2012) used data recorded from 1,067 bridge I-girders as part of a study sponsored by the Minnesota Department of Transportation (MNDOT) and determined that camber at release was over-predicted by 74% and camber at erection was over-predicted by 86%. The over-predictions resulted primarily from variations in the concrete compressive strength and consequently modulus of elasticity used in the calculations. Field testing was performed on a limited number of concrete samples to determine strength and stiffness. In addition, 14 various sized girders were monitored for time-dependent deflections from time of fabrication through shipment. Time-dependent effects on long-term camber were evaluated using an analytical model which was used to validate a multiplier based prediction model for use by MNDOT. It was concluded that their multipliers greatly improved pre-erection camber predictions. The study also

gave recommendations on fabrication methods to reduce the variability of girders produced. By using the combined multipliers and guidelines for fabrication it was suggested that variability in camber could be reduced to less than $\pm 15\%$.

A.20. Schrantz (2012)

Schrantz (2012) developed a Visual Basic based computer program to predict long-term camber in prestressed concrete beams. The computer program takes into account creep, and shrinkage effects in addition to modulus of elasticity and was validated using measured strain and camber data from Boehm (2008), Levy (2007), and Stallings et al. (2003). The computer program was used to evaluate various prediction models in terms of their capability to accurately predict camber. The work of Schrantz (2012), was later refined by Johnson (2012), and also used by Mante (2016) to develop a prediction method based on an incremental time-step approach.

A.21. Johnson (2012)

Johnson (2012) collected camber data on 28 bulb-tee girders constructed with self-consolidating concrete supplied for an Alabama Department of Transportation (ALDOT) bridge project. Predictions of time-dependent camber were compared to actual measurements taken over the course of the project. Material testing was conducted to determine time-dependent properties. The following conclusions were drawn: (1) when considering the bottom-flange of girders, time-dependent strain predictions were reasonably accurate for all creep and shrinkage models used, (2) concrete strains were over-estimated at later girder ages, (3) the effective prestressing force was overpredicted in the first months after prestress transfer and underpredicted at later ages, (4) measured mid-span camber was usually less than predicted for ages up to 200 days, and (5) none of the time-dependent models used predicted camber growth well. The study demonstrated that

improved methods for predicting time-dependent camber growth needed to be developed for ALDOT, which was later addressed by Mante (2016).

A.22. Precast/Prestressed Concrete Institute Committee on Bridges (2012)

The PCI Committee on Bridges (2012) conducted a study to evaluate the construction tolerances for camber in prestressed concrete bridge girders specified by then current PCI standards. Camber values were collected for 1,835 bridge girders from eight different states for analysis. Using the measured and predicted camber values of these girders, changes to the allowable camber tolerances at release were recommended. The Committee recommended increasing the tolerance limits to match historical data. The percent of predictions that were within the new tolerance limits increased to 90% compared to the 66%, which was based on the old tolerance limits.

A.23. Bažant et al. (2012)

Bažant et al. (2012) investigated long-term deflections in a segmental prestressed concrete box girder bridge located in Palau, . The span of the bridge was 791 ft; the bridge was erected in 1977 and collapsed in 1996 after failed attempts to repair it. At the age of 18 years, the bridge was found to have a measured mid-span camber 5.3 ft greater than the design camber. Results from finite element models were compared to time-step predictions using creep and shrinkage models from then-current provisions of American Concrete Institute, Japan Society of Civil Engineers, Comité Euro-International du Béton, and Gardner and Lockman (2000) recommendations. These prediction models were found to provide inaccurate estimates of the effects of long-term creep and shrinkage. For 18-year deflection estimates, the models were 50–77% lower than measured deflections and yielded unrealistic shapes of the deflection history. They also predicted the 18-year prestress loss to be 46–56% less than measured. The analysis done by Bažant et al. (2012)

emphasized the importance of accurate calculations for time-dependent properties and prestress losses. Future work was recommended to develop newer, more accurate prediction models.

A.24. Storm et al. (2013)

Storm, Rizkalla, and Zia (2013) performed a study on various factors affecting prediction of camber, with a focus on the factors related to girder fabrication. A field study was conducted using 382 pretensioned concrete girders from nine different states. Camber measurements from the 382 girders were collected and analyzed from four different stages: (1) immediately after prestress transfer, (2) at the beginning of storage, (3) prior to shipping, and (4) after erection. A laboratory study was also conducted to obtain compressive strength, modulus, and unit weight of concrete at various ages. It was concluded that: (1) Camber predictions should account for the typically higher compressive strength at prestress transfer and service compared with specified values, (2) camber predictions should consider debonding and transfer length, especially for girders with long debonding lengths, (3) curing method can significantly affect camber at time of prestress, (4) Measured camber can vary significantly among girders that are identical in their design even for girders cast at the same time on the same casting bed, this is due in part to multiple batches of concrete being used for a single casting and (5) A refined method (time-step method) provides the most accurate camber predictions for most girder types and curing methods, while the approximate method (PCI multiplier) generally overestimates camber at erection, but can still be useful for preliminary estimates.

A.25. Mahmood (2013)

Many of the studies discussed so far were concerned with the topic of how to accurately predict camber. Another topic of interest is how to control camber in cases when camber is initially higher than predicted and consequently grows to higher than predicted values. Mahmood (2013)

explored the feasibility of controlling camber in simply-supported prestressed concrete bridge girders by using post-tensioned strands. This theoretical study also considered the reduction in load capacity of the girder. The research demonstrated that using the post-tensioned strands to control camber was proved to be a feasible option and that the method resulted in minimal reduction to load-carrying capacity. For instance, an AASHTO Type IV girder experienced a load-carrying capacity reduction of 2.9 % per 100 kips of jacking force. Recommendations for future studies include: (1) using other girder types to further investigate the effects of such a camber control method; and (2) conduct a study on the practicality of the proposed method.

A.26. He (2013)

He (2013) conducted a study for the Iowa Department of transportation on improving prediction of long-term camber in prestressed concrete girders. Seven concrete mixes with regional properties were tested for modulus of elasticity, creep and shrinkage. The recorded properties were used to compare several prediction techniques to the camber measured in 26 prestressed high-performance concrete girders. Conclusions from the work include: (1) the prediction method in current use over predicts camber by 30% in long-span bridges, (2) modulus of elasticity in concrete varied by 205 between the AASHTO (2012) predicted values and measured values, (3) sealed concrete specimens tended to represent the creep and shrinkage behavior of the full scale prestressed girder better than unsealed specimens, (4) when calculated by gross section properties instead of transformed section properties, girder camber was on average 13 percent higher, (5) the time-steps method implemented in this study predicted camber within 25 percent accuracy, and (6) about 50% of ultimate camber growth had occurred one year after girder production.

A.27. Nervig (2014)

Nervig (2014) performed a study funded by the Iowa Department of Transportation (IDOT), focusing on improving camber at release estimates for concrete bridge girders. The camber at release of 105 prestressed concrete beams was measured and compared to predicted values. Inconsistent field measurement methods and inaccurate estimates of material properties were found to be the main cause for discrepancies in camber at release estimates. To increase the accuracy of camber at release estimates, Nervig (2014) recommended that: (1) AASHTO LRFD (2010) equation for transfer length should be used in calculations, (2) using AASHTO LRFD (2010) modulus of elasticity equation with an accurate release strength and unit weight will improve camber predictions, (3) designers should increase the design release strength by 40% and 10% for beams with designed release strength of 4500-5500 psi and 6000-8500 psi, respectively, (4) actual prestressing force should be used for calculations, as well as accurate prestressing losses, and (5) sacrificial prestress strands should be accounted for as they can affect camber by as much as 6.46%.

A.28. Hofrichter (2014)

Provided that many previous studies emphasized the use of measured material properties to obtain more accurate estimates of camber, Hofrichter (2014) conducted a study to establish a relationship between specified and measured properties so that this information can be used during the design phase. Compressive strength data from more than 1,900 prestressed concrete beams was collected. The beams were fabricated in Alabama and consequently the collected data was regionally exclusive to Alabama. The collected data was used to refine methods for predicting average concrete compressive strength at prestress transfer and at 28 days. It was found that release strengths and 28-day strengths are often much higher than the specified strengths, more so for the

28 day strengths. A strength prediction equation was proposed which incorporated an assumed aggregate factor. The proposed equation resulted in better prediction of compressive strength and modulus of elasticity.

A.29. Isbilibroglu (2014)

Isbilibroglu (2014) expanded on the work of Schrantz (2012) and Johnson (2012) to finalize a camber prediction software that incorporated an incremental time-step method. Experimental research produced by Johnson (2012), Schrantz (2012), Boehm (2008), Stallings et al. (2003), and Levy (2007), were used in validation of the program. Isbilibroglu (2014) worked to improve the camber prediction software by: (1) Allowing the user to import and export files, creating an easier-to-use interface, (2) refining material prediction models and utilizing newer design code parameters; and (3) including recommendations made by Hofrichter (2014) and Keske (2014).

A.30. Nguyen (2014)

Nguyen (2014) explored methods to predict camber in precast, prestressed concrete girders, with a focus on the effects of temperature on camber during curing and in while in service. Historical data from Rosa et al. (2007) was used, as well as new data acquired from the fabrication of nine girders for the Alaska Way Viaduct project in Tacoma, WA. Research focused on using both fabrication camber and field-measured camber to calibrate models predicting camber. The effect of daily temperature variations on girder camber was also studied using recorded temperature histories at release and service. Two models were developed to predict daily camber changes under solar radiation, which correlated well with the ambient temperature data collected during the research. Some notable conclusions from the study include: (1) using actual, rather than design compressive strength will improve camber predictions, especially for long-term camber, (2) camber estimates using the elastic modulus from the AASHTO 2006 and ACI 363

recommendations tended to overestimate the measured camber, (3) the NCHRP 496 method of calculating the elastic modulus led to better predictions of camber, (4) girder camber is significantly affected by daily variations in the girder temperature distribution and the effects should be considered in estimating the camber, and (5) thermal effects are potentially major causes of the discrepancies between measured and predicted release camber.

A.31. Honarvar et al. (2015)

Honarvar et al. (2015) conducted a study funded by the Iowa Department of Transportation (IDOT) to address discrepancies between the design and measured camber in precast pretensioned concrete beams. IDOT's current practice at the time of study was to perform elastic analysis and apply Martin's multipliers (1977) to estimate the camber at release and erection. This method was observed by IDOT to frequently over predict camber. The study included material testing for creep and shrinkage of both normal and high-performance concrete specimens, as well as identified other variables affecting camber. Long-term camber was evaluated using finite element analysis and time-step methods. The study resulted in new multipliers for camber that, when used with accurate camber at release measurements greatly increase the accuracy of predicted camber.

A.32. Keraga (2016)

Keraga (2016) compared predicted and measured camber for a variety of prestressed concrete box girder bridges and recommended a set of multipliers including lower and upper bounds. The recommended multipliers are lower than those recommended by PCI and Martin (1977) and are 1.65 for the prestress camber and 1.70 for the self-weight deflection.

A.33. Menkulasi et al. (2018)

Menkulasi et al. (2018) performed a study on the restraint moments developed in simple span bridges made continuous for live loads. Long-term beam camber was also examined by comparing predicted and measured values as camber growth affects the magnitude of restraint moments after continuity has been established. It was concluded that camber predictions based on the AASHTO (2017) and CEB MC90-99 models matched rather well with measured values, while the ACI 209R-92 model resulted in slightly higher camber values, and the GL2000 and B3 models overestimated beam camber significantly.

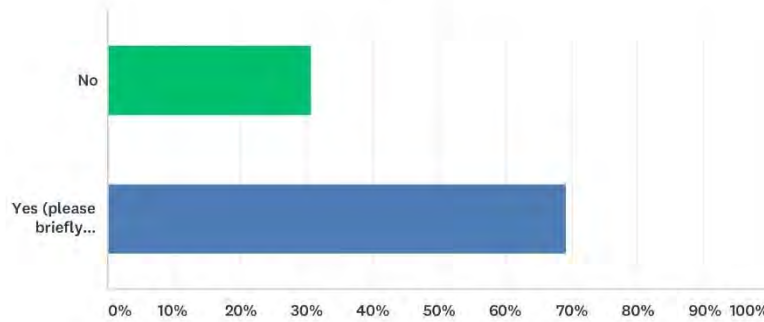
A.34. Mante et al. (2019)

Mante et al. (2019) conducted a comprehensive camber prediction study sponsored by the Alabama Department of Transportation (ALDOT). The research objective was to provide practical recommendations to improve camber predictions for precast, prestressed concrete bridge girders. The study incorporated concrete materials data from nearly 2000 girder production cycles among four regional producers, the data was used to develop regional calibration factors for properties essential in predicting camber growth, including compressive strength, modulus, creep, and shrinkage. A standard incremental time-step analysis software (ALCAMBER V1.0) was developed and utilized for conducting a parametric study. Using the developed regional calibration factors resulted in the elimination of approximately 80% of the prediction error associated with current camber prediction practices within the region. Current design practices resulted in a mean over prediction in camber of 68% whereas regionally calibrated prediction models had a mean over prediction of approximately 10%. Using expected vs. specified compressive strength values, aggregate correction factors for modulus of elasticity, and incremental time-step method, was shown to have the most effective improvement in predictions.

APPENDIX B: Prestressed Concrete Beam Camber Questionnaire

Q1 Have you ever experienced any camber related problems with prestressed concrete bridge beams that are often beyond normal construction tolerances?

Answered: 39 Skipped: 4



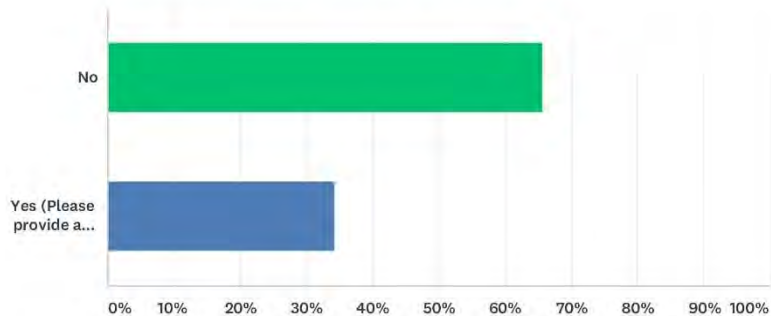
ANSWER CHOICES		RESPONSES	
No		30.77%	12
Yes (please briefly describe)		69.23%	27
TOTAL			39

#	YES (PLEASE BRIEFLY DESCRIBE)	DATE
1	We have had instances where cambers greatly exceeded the normally expected values. In rare cases, these girders had to be re-cast because they could not be made to fit into the bridge elevation profiles using normal mitigation measures. Other more common cases involve girders fabricated at the same time but with large variations in storage duration due to project phasing. The second phase girders usually arrive with higher cambers than the 1st phase girders but no adjustment of the bridge grades can be made to accommodate the 2nd phase girders	9/12/2019 4:44 AM
2	Occasionally, actual camber exceeds the provided camber strip thickness between the top of girder and bottom of deck. When this occurs, an adjustment to the finished grade profile is required to maintain minimum deck thickness.	8/26/2019 10:55 AM
3	Camber that is over tolerance as well as the opposite, situations where the cast deck results in beams having negative camber.	8/21/2019 7:12 AM
4	Note that all answers herein assume your questions are regarding pretensioned beam members (p/t members are not addressed). In the early 2000s, it was observed that cambers were typically much lower than predicted.	8/12/2019 11:40 AM
5	The actual camber was less than the predicted camber. Thus, the erected beams once the deck was placed resulted in zero to negative camber. We are monitoring a few bridges built to this situation.	8/12/2019 6:42 AM
6	There has been camber variation greater than the maximum 1 inch tolerance per MNL-116 Appendix B, B-10 I-Beam (Girder) or Bulb Tee-Girder.	8/9/2019 9:09 AM
7	Trey Carroll, thcarroll1@ncdot.gov, 919-707-6465, submitting for North Carolina DOT. Before NCDOT's camber research project was implemented camber predictions (particularly for cored slabs and box beams) were not reliable.	8/7/2019 7:45 AM
8	Sometimes camber between adjacent beams is not similar and camber calculated by AASHTO equations is not the same as the camber in the fabricator's shop.	8/7/2019 3:27 AM

9	The primary issue appears to be with prestressed beams that sit at the fabricator unloaded for extended (6 mo +) period prior to deck placement. This is especially problematic for project built in phases when the beams are all cast at once. The camber growth between phases is critical.	8/6/2019 9:15 AM
10	The camber of the beams is sometimes inconsistent between abutting beams and the value calculated by AASHTO equations is lower than what is seen at the precast plant.	8/6/2019 6:22 AM
11	Most bridges are ok, but at times there is variance between girders.	8/6/2019 5:01 AM
12	For the most part, projects seem to be within tolerance, but can occasionally have variance between girders.	8/6/2019 4:55 AM
13	Problems with prestressed concrete bridge members are typically experienced where members are cast more than three months prior to planned erection. Contractors are assumed to schedule this early fabrication typically to ensure members are available for their planned erection schedule and, for multi-stage projects, to achieve potential cost savings or ease of fabrication. In the case of multi-stage projects, the period between staged erection could be 6 months or in excess of one year. Problems with prestressed concrete bridge members are also typically experienced where a member depth is selected in the lower range of applicability and a larger number of strands and higher concrete strength are used accordingly.	8/5/2019 12:10 PM
14	It has been difficult sometimes to get the predicted camber	8/2/2019 9:04 AM
15	Our primary issue is the discrepancy between the estimated time and actual time of girder placement after strand are released. Girders that are estimated to camber up may actually sag when construction is accelerated.	8/2/2019 8:13 AM
16	We have had over cambering occur due to beams not being put in service soon enough such construction delays	8/2/2019 7:48 AM
17	Camber growth that compromises the deck thickness.	8/2/2019 4:58 AM
18	In long-span (greater than 120-feet) bulb-tee girders (such as the SCDOT Modified 72" Bulb Tee), we have had multiple cases where the camber did not grow as much as the PCI equations predict. In these examples the concrete strength was 10ksi (min.).	8/1/2019 3:27 PM
19	We had a beam with zero camber that we rejected due to concerns over loss of prestress. The beam was rejected.	8/1/2019 12:47 PM
20	Beams that have excessive camber and beams have less than designed for camber.	8/1/2019 3:46 AM
21	Most of the time, large cambers were observed where debonded strands design was utilized. Other occasions camber growth was an issue. Long term issues were related to girders designed with no tension allowed at final condition	7/31/2019 1:05 PM
22	In the past, we have had issues with cambers not matching the values that were calculated using the AASHTO equations. In response, we developed our own equations that are included in our bridge design manual.	7/31/2019 12:44 PM
23	Many factors affect cambers, such as concrete mix/strength/quality/curing, rest time (from fabrication to installation), prestressing force, section properties, formulas for calculating camber and so forth. We added Camber Management Plan in the specifications to mitigate this issue. We also adjusted the tolerances.	7/31/2019 12:20 PM
24	Most issues are from beam camber growth from sitting in the fabricator's yard too long.	7/31/2019 11:42 AM
25	Have camber issues fairly often, especially when the beams are made far in advance.	7/31/2019 11:36 AM
26	A couple of bridges over the last 5 years have less camber than estimated. The constructed bridges essentially have zero or negative camber. We are monitoring (elevation survey) these couple of bridges to determine if we are continuing to lose camber. To date these bridges are functioning adequately.	7/31/2019 11:35 AM
27	Increased amount of camber that caused fit up issues with adjacent beams on a high skewed bridge	7/31/2019 11:22 AM

Q2 Have you conducted or sponsored any research related to prestress losses, camber, and deflections?

Answered: 38 Skipped: 5



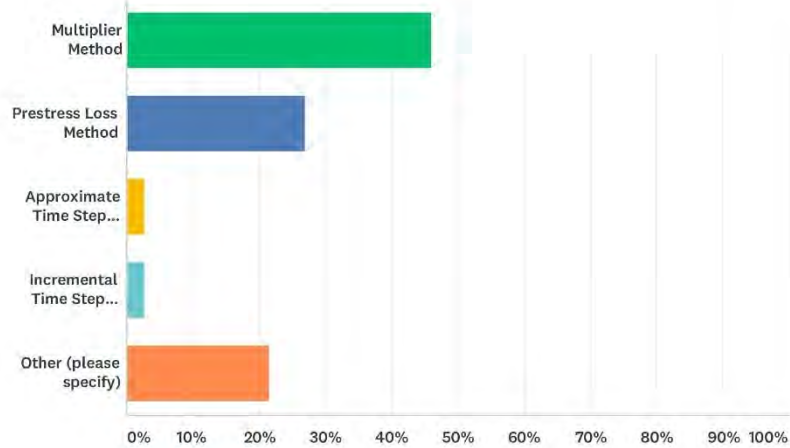
ANSWER CHOICES	RESPONSES	
No	65.79%	25
Yes (Please provide a reference to your report and indicate whether the results of the research have been implemented into your state's practices)	34.21%	13
TOTAL		38

#	YES (PLEASE PROVIDE A REFERENCE TO YOUR REPORT AND INDICATE WHETHER THE RESULTS OF THE RESEARCH HAVE BEEN IMPLEMENTED INTO YOUR STATE'S PRACTICES)	DATE
1	Sinno, Raouf. The Time-Dependent Deflections of Prestressed Concrete Bridge Beams, Ph.D. Dissertation, Texas A&M University, College Station, Texas, January 1968. Kelly, D. J., Bradberry, T. E., and Breen, J. E. "Time-Dependant Deflections of Pretensioned Beams," Research Report 381-1, Research Project 3-5-84-381, Center for Transportation Research, Bureau of Engineering Research, The University of Texas at Austin, Austin, TX (August 1987) 211 pp. URL: https://library.ctr.utexas.edu/digitized/texasarchive/phase2/381-1-CTR.pdf We used this research to design beams, until we adopted PGSuper for prestressed concrete girder design. We now use the same method used by the program.	9/12/2019 4:44 AM
2	Report No. UT-09.10: UDOT Calibration of AASHTO's New Prestress Loss Design Equations Link: https://www.udot.utah.gov/main/uconowner.gf?n=7904804259161866	8/26/2019 10:55 AM
3	A small internal study resulted in changing camber multipliers from the PCI values to a value of 1.5. We also sponsored a camber study to confirm/change the camber multiplier. The result was implementation of a slightly modified approach using a multiplier of 1.4. Report is "Validation of Prestressed Concrete I-Beam Deflections and Camber Estimates", which can be found at: https://www.lrrb.org/media/reports/201216.pdf	8/12/2019 11:40 AM
4	Keraga, Cody. Development of Camber Multipliers for Precast Prestressed Box Girders. 2016, digital.auraria.edu/content/AA/00/00/51/37/00001/Keraga_ucdenver_0765N_10686.pdf . This has been implemented into the state's practice by allowing a camber tolerance for precast girders. This tolerance is used in the design.	8/9/2019 9:09 AM
5	NCDOT sponsored research titled "Predicting Camber, Deflection, and Prestress Losses in Prestressed Members" and results from this report have been successfully implemented. https://connect.ncdot.gov/resources/Structures/Pages/Structure-Resources.aspx	8/7/2019 7:45 AM
6	We looked at girder camber values of 800 girders and compared those values to the theoretical value at release. From this, we require a multiplier of 1.4.	8/6/2019 5:01 AM

7	We looked at the camber of about 800 girders in Wisconsin and compared values to the design values. From this study, we implemented a camber multiplier of 1.4 to be applied to the theoretical camber at release.	8/6/2019 4:55 AM
8	Cousins, T.E., and Gomez, J.P. Investigation of Long-term Prestress Losses in Pretensioned High Performance Concrete Girders, VTRC Report 05-CR20. Virginia Transportation Research Council, Charlottesville, 2005 Cousins, T.E., and Nassar, A.J. Investigation of Transfer Length, Development Length, Flexural Strength, and Prestress Losses in Lightweight Prestressed Concrete Girders, VTRC Report 03-CR20. VTRC Report 05-CR20. Virginia Transportation Research Council, Charlottesville, 2003 In-formal research was conducted in support of specification updates related to camber, but a report was not developed.	8/5/2019 12:10 PM
9	We have monitored lots of girder production to compare with predicted. NDOT ended up using k factor of 0.9 instead of 1.0 in calculating the section modulus	8/2/2019 9:04 AM
10	http://www.virginiadot.org/vtrc/main/online_reports/pdf/05-cr20.pdf	7/31/2019 12:20 PM
11	https://fdotwww.blob.core.windows.net/sitefinity/docs/default-source/content/structures/structuresresearchcenter/final-reports/bd545_07.pdf?sfvrsn=d665c214_0	7/31/2019 12:00 PM
12	Improving the Accuracy of Camber Predictions for Precast Pretensioned Concrete Beams, IHRB Project TR-625, Sri Sritharan, July 2015. Implemented to a limited degree. Additional implementation likely with future beam standards update.	7/31/2019 11:42 AM
13	Research project to measure prestress loss of decked bulb-T girders has recently begun by Dr. Il-Sang Ahn at the University of Alaska Fairbanks.	7/31/2019 11:38 AM

Q3 What method do you use to predict prestressed concrete beam camber?

Answered: 37 Skipped: 6

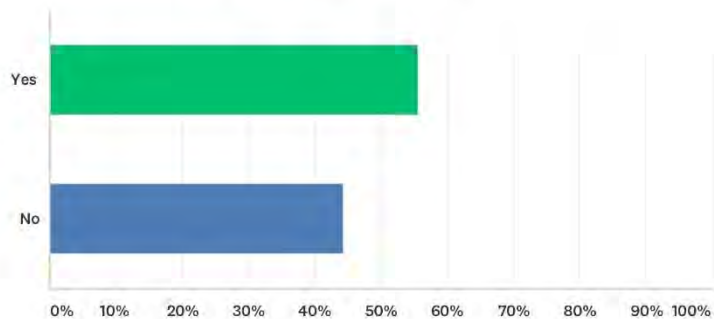


ANSWER CHOICES	RESPONSES	
Multiplier Method	45.95%	17
Prestress Loss Method	27.03%	10
Approximate Time Step Method	2.70%	1
Incremental Time Step Method	2.70%	1
Other (please specify)	21.62%	8
TOTAL		37

#	OTHER (PLEASE SPECIFY)	DATE
1	WSDOTs method as programmed in PGSuper.	9/12/2019 4:44 AM
2	The use of FDOT research that quantifies elastic shortening and shrinkage-related losses.	8/21/2019 7:13 AM
3	NCDOT Refined Method for Camber (uses a modified approximate time step method) See page 70 of the research report.	8/7/2019 7:46 AM
4	Please refer to ODOT BDM Section 302.5.2.3 at the following link: http://www.dot.state.oh.us/Divisions/Engineering/Structures/standard/Bridges/BDM/2019_BDM_07-19-19.pdf	8/6/2019 9:17 AM
5	Multiplier method with all factors set to 1.0	8/2/2019 4:59 AM
6	State specific equations	7/31/2019 12:44 PM
7	University of Florida Method. or the PCI multiplier method.	7/31/2019 12:02 PM
8	None of the above	7/31/2019 11:37 AM

Q4 Does this method explicitly consider the effects of creep and shrinkage?

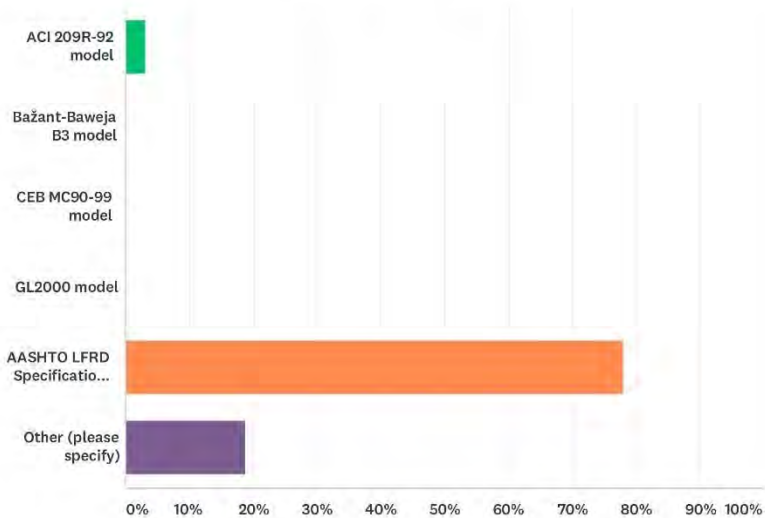
Answered: 9 Skipped: 34



ANSWER CHOICES	RESPONSES	
Yes	55.56%	5
No	44.44%	4
TOTAL		9

Q5 What model do you use to account for creep and shrinkage effects?

Answered: 32 Skipped: 11



ANSWER CHOICES	RESPONSES	
ACI 209R-92 model	3.13%	1
Bažant-Baweja B3 model	0.00%	0
CEB MC90-99 model	0.00%	0
GL2000 model	0.00%	0
AASHTO LFRD Specifications model	78.13%	25
Other (please specify)	18.75%	6
TOTAL		32

#	OTHER (PLEASE SPECIFY)	DATE
1	?	9/19/2019 5:16 AM
2	In PGSuper - Modulus of elasticity at transfer is computed using a modified form of AASHTO LRFD Equation 5.4.2.4-1 using f_{ci} . This equation is modified with the factors K_1 and K_2 as defined in NCHRP Report 496. Creep and shrinkage are computed with modified versions of AASHTO LRFD Equations 5.4.2.3.2-1 and 5.4.2.3.3-1. These equations are modified with K_1 and K_2 factors as defined in NCHRP Report 496. For girders having temporary top strand to control stability during transportation, the elastic shortening losses and K_{id} and K_{df} transformed section coefficients are modified to compute the shortening, creep, and shrinkage losses in the permanent strands. Modifications account for the fact that the temporary strands raise the CG of the prestressing force while the majority of the creep and shrinkage losses are taking place.	9/12/2019 4:45 AM
3	Creep and shrinkage are accounted for by way of calculation in the FDOT Prestressed Beam Program.	8/21/2019 7:14 AM
4	For camber, creep and shrinkage are not explicitly calculated. Instead, they are lumped into the multiplier.	8/12/2019 11:42 AM
5	Just Creep	8/12/2019 6:42 AM
6	Do not explicitly consider.	8/5/2019 12:12 PM

Q6 Do you have a procedure or a tool that you use to predict camber and displacement at a given time after the erection of the deck and after the opening of the bridge to traffic?

Answered: 30 Skipped: 13

#	RESPONSES	DATE
1	No	9/12/2019 4:46 AM
2	Multipliers in UDOT Structures Design and Detailing Manual 14.6.4.	8/26/2019 11:30 AM
3	N/A.	8/21/2019 7:16 AM
4	No.	8/12/2019 11:48 AM
5	No	8/12/2019 6:44 AM
6	Multipliers in Bentley LEAP Concrete Bridge. There is not a specific tool used to predict time dependent camber and displacements.	8/9/2019 9:19 AM
7	Final Camber Prediction = Girder Alone - Superimposed Dead Load	8/7/2019 7:46 AM
8	No.	8/7/2019 3:32 AM
9	Please see ODOT C&MS Section 511.07 for construction procedure and ODOT C&MS Table 515.16-5 for tolerances at the following link: http://www.dot.state.oh.us/Divisions/ConstructionMgt/OnlineDocs/Specifications/2019CMS/2019_CMS_07192019_for_web_letter_size.pdf	8/6/2019 9:25 AM
10	No.	8/6/2019 6:36 AM
11	No	8/6/2019 5:13 AM
12	No.	8/5/2019 12:15 PM
13	We would use the same equations that we use to predict final camber after deck placement (90 days).	8/2/2019 9:09 AM
14	No. We use AASHTO time dependent method	8/2/2019 9:08 AM
15	No	8/2/2019 8:16 AM
16	No.	8/2/2019 4:48 AM
17	No	8/1/2019 3:30 PM
18	No	8/1/2019 6:48 AM
19	NO	8/1/2019 3:59 AM
20	No	8/1/2019 2:27 AM
21	Leap Conspan	7/31/2019 1:48 PM
22	Take measurements at various stages including pre installation of the risers	7/31/2019 1:46 PM
23	No	7/31/2019 1:09 PM
24	No.	7/31/2019 12:48 PM
25	No.	7/31/2019 12:23 PM
26	Long term camber (after composite) is not checked.	7/31/2019 12:06 PM
27	No	7/31/2019 11:51 AM
28	We use the deflection multiplier method. We have multipliers for short-term and long-term predictions but these are only used as a check of fabricator predicted values. Girder fabricators working for the Alaska DOT&PF have the capability to sag the forms to compensate for predicted camber.	7/31/2019 11:43 AM
29	We have an inhouse program, PSLRFD	7/31/2019 11:42 AM
30	No	7/31/2019 11:39 AM

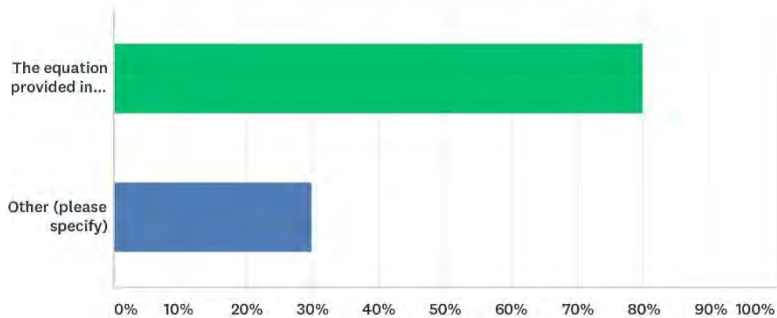
Q7 What is the range of 28-day compressive strength and compressive strength at release that you typically specify for prestressed concrete beams?

Answered: 31 Skipped: 12

#	RESPONSES	DATE
1	f_{ci} – 4 ksi to 6 ksi f_c – 5 ksi to 8.5 ksi	9/12/2019 4:46 AM
2	28-day compressive strength = 5.0 to 8.5 ksi Compressive strength at release = 5.0 to 7.5 ksi	8/26/2019 11:30 AM
3	Ranges of compressive strength are typically 8.5 ksi, but may be occasionally higher even to 10 ksi.	8/21/2019 7:16 AM
4	f_{ci} range is from 4.5 ksi to 7.5 ksi f_c range is from 5.0 ksi to 9.0 ksi	8/12/2019 11:48 AM
5	Varies based on design. Typically, we see 6.8 ksi before stress transfer and see 8 ksi for 28-day design compressive strengths.	8/12/2019 6:44 AM
6	f_{ci} =6,500psi f_c =8,500psi	8/9/2019 9:19 AM
7	28-Day = 5ksi-10ksi Release = 4ksi-8ksi	8/7/2019 7:46 AM
8	The release strength typically dictates the 28 day compressive strength. We specify the minimum release strength to satisfy the top of the end of the beams stresses. Release is approximately 80% of final so if we need 4800 psi at release we specify 6000 for final. Anything that needs 4800+ to 6400 psi at release will be specified as 8000 psi for final/	8/7/2019 3:32 AM
9	f_{ci} = 5 ksi f_c = 7 ksi These are ODOT minimums as specified in the ODOT Standard Bridge Drawings PSBD-2-07 and PSID-1-13. Designers may increase these with prior written approval of our fabricators. Standards may be found at the following link: http://www.dot.state.oh.us/Divisions/Engineering/Structures/standard/Bridges/Pages/StandardBridgeDrawings.aspx	8/6/2019 9:25 AM
10	We typically will specify the strength that we need for release to avoid cracking at the top ends of the girders. The final strength is determined so that the release is 80% of that. So if we needed 4800 at release we would specify 6000. if we needed a release strength between 4800 and 6400 we would specify 8000.	8/6/2019 6:36 AM
11	f_{ci} = 6400 to 6800 psi for girders with f_c = 8000 is typical.	8/6/2019 5:13 AM
12	0.8 f_c (e.g., release typically minimum of 6.4 ksi for 8 ksi concrete)	8/5/2019 12:15 PM
13	6.5 ksi (release) 8.0 ksi (final)	8/2/2019 9:09 AM
14	8 ksi- 12 ksi	8/2/2019 9:08 AM
15	no range - specify strength at release and strength 28-day	8/2/2019 8:16 AM
16	min	8/2/2019 7:49 AM
17	Usually it's 6,000 psi, but we have gone up to 8,000 psi.	8/2/2019 4:48 AM
18	6ksi - 10 ksi	8/1/2019 3:30 PM
19	10 ksi	8/1/2019 6:48 AM
20	Range for 28 days varies between 5 Ksi to 8 Ksi Range at release varies between 4 Ksi and 7 Ksi. 1 to 1.5 Ksi difference between release and 28 days.	8/1/2019 3:59 AM
21	Compressive strength at release is approximately 90% of 28-day strength.	8/1/2019 2:27 AM
22	8,000 psi/6,000 psi	7/31/2019 1:48 PM
23	4500 RELEASE 6000 -28 DAYS FOR PILES ONLY 6000 PSI TO 6500 PSI RELEASE 8500 PSI - 28 DAYS RELEASE 7500 PSI AND 10000 PSI -28 DAYS	7/31/2019 1:46 PM
24	28 day 6000 at release 5000	7/31/2019 1:09 PM
25	6-8ksi for 28-day strength, 80% for release	7/31/2019 12:48 PM
26	5,000psi to 10,000psi	7/31/2019 12:23 PM
27	Release 6-6.8 ksi. 28 day strength 8.5 ksi	7/31/2019 12:06 PM
28	Release 4.5 ksi to 8 ksi Final 5 ksi to 9 ksi	7/31/2019 11:51 AM
29	28 day f_c ~ 7500 psi to 8500 psi with test results typically between 9000 psi and 12000 psi 16-hour f_{ci} ~ 6000 psi to 7500 psi with test results typically between 7000 psi and 9000 psi	7/31/2019 11:43 AM
30	8 ksi 6.8 ksi	7/31/2019 11:42 AM
31	28 day 6-8 ksi At release 4-7 ksi	7/31/2019 11:39 AM

Q8 How do you determine modulus of elasticity when conducting camber calculations? (choose all that apply)

Answered: 30 Skipped: 13



ANSWER CHOICES

RESPONSES

The equation provided in AASHTO LRFD Bridge Design Specifications	80.00%	24
Other (please specify)	30.00%	9
Total Respondents: 30		

#	OTHER (PLEASE SPECIFY)	DATE
1	TxDOT policy is to set all concrete moduli to 5000 ksi for both stress and deflection calculations and to indicate on the plans that the deflections shown are calculated assuming an E'c of 5000 ksi.	9/12/2019 4:46 AM
2	We use the ACI 363 equation: $E_c = 1265 \times (f'_c)^{0.5} + 1000$ where f'_c and E_c are in ksi.	8/12/2019 11:48 AM
3	modulus of elasticity of beam concrete at transfer	8/12/2019 6:44 AM
4	AASHTO LRFD Bridge Design Specification equation (7th Edition) $K1=0.85$	8/7/2019 7:46 AM
5	$(5500 \times \text{SQRT}(f'_c)) / (\text{SQRT}(6))$ This is based on historical data.	8/6/2019 5:13 AM
6	We use the AASHTO LRFD equation that was removed around 2015. $E_c = 33000K1(wc 1.5)(f'_c 0.5)$	8/2/2019 9:09 AM
7	we use K of 0.9 instead of 1.0 because of Nebraska aggregate	8/2/2019 9:08 AM
8	Provided by the contractor	7/31/2019 1:46 PM
9	$K=1.0$	7/31/2019 12:06 PM

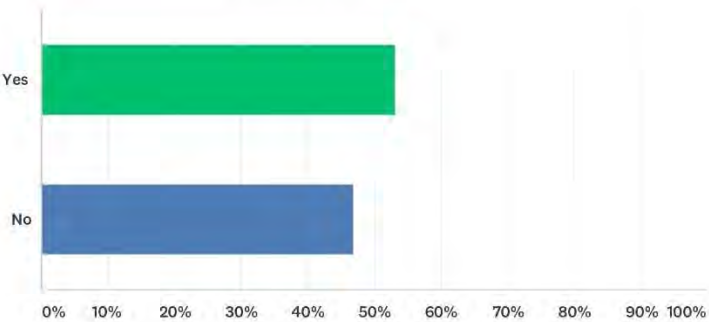
Q9 If the equation provided in AASHTO LRFD Bridge Design Specifications is used to determine the modulus of elasticity, do you use the specified compressive strength at release and the 28-day compressive strength from the plans or some other value?

Answered: 29 Skipped: 14

#	RESPONSES	DATE
1	N/A	9/12/2019 4:46 AM
2	Use specified compressive strength at release and the 28-day compressive strength from the plans	8/26/2019 11:30 AM
3	Compressive strength at release and 28-day as stated in the Plans.	8/21/2019 7:16 AM
4	We use the compressive strengths given in the bridge plan for calculating E_c in the equation above.	8/12/2019 11:48 AM
5	$f_c = f_{ci}$	8/12/2019 6:44 AM
6	Usually the specified compressive strengths are used. The designer has the option of using actual average values of $f_{ci}=8.5\text{ksi}$ and $f_c=12.5\text{ksi}$	8/9/2019 9:19 AM
7	Specified release strength is increased by 25% and the specified 28-day strength is increased by 45%	8/7/2019 7:46 AM
8	We use the 28 day compressive strength.	8/7/2019 3:32 AM
9	Plans	8/6/2019 9:25 AM
10	We use the final design strength. This is usually lower than what is being provided by the precaster. The precaster tries to turn their beds every 24 hours so they will use a higher strength mix to get the design release strength earlier.	8/6/2019 6:36 AM
11	NA	8/6/2019 5:13 AM
12	Plans	8/5/2019 12:15 PM
13	Plan Values	8/2/2019 9:09 AM
14	from the plans	8/2/2019 9:08 AM
15	From plans	8/2/2019 8:16 AM
16	We use the specified compressive strength at release and the 28-day compressive strength from the plans.	8/2/2019 4:48 AM
17	Specified compressive strength at release and the 28-day compressive strength.	8/1/2019 3:30 PM
18	Yes	8/1/2019 6:48 AM
19	We use specified compressive strength at release and 28-day compressive strength from the plans.	8/1/2019 3:59 AM
20	Use 28 day compressive strength	8/1/2019 2:27 AM
21	Values from the plans.	7/31/2019 1:48 PM
22	we use release and 28 day.	7/31/2019 1:09 PM
23	Yes.	7/31/2019 12:48 PM
24	From the plans.	7/31/2019 12:23 PM
25	Specified 28-day strength in the plans.	7/31/2019 12:06 PM
26	At release, although we have adopted a procedure to adjust based on expected strengths since they are typically higher.	7/31/2019 11:51 AM
27	Yes, we use the specified (not expected) f_{ci} and assume that the variability is included in the deflection multipliers. We use the E_{ci} modulus as the basis for deflection.	7/31/2019 11:43 AM
28	From the plans	7/31/2019 11:42 AM
29	release and the 28-day	7/31/2019 11:39 AM

Q10 Do you require a specific curing technique for prestressed concrete beams?

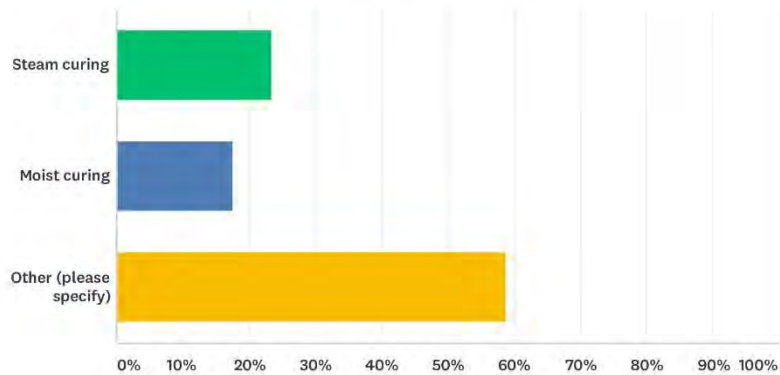
Answered: 32 Skipped: 11



ANSWER CHOICES	RESPONSES	
Yes	53.13%	17
No	46.88%	15
TOTAL		32

Q11 Which curing technique do you specify?

Answered: 17 Skipped: 26



ANSWER CHOICES	RESPONSES	
Steam curing	23.53%	4
Moist curing	17.65%	3
Other (please specify)	58.82%	10
TOTAL		17

#	OTHER (PLEASE SPECIFY)	DATE
1	any curing technique is acceptable provided all exposed surfaces remain wet until release strength is achieved. Curing compound is not allowed on the top surface girders. Standard Specifications for Construction and Maintenance of Highways, Streets and Bridges, Item 424 - ftp://ftp.dot.state.tx.us/pub/txdot-info/des/spec-book-1114.pdf - This specification allows steam, moist, barrier, and wet mat. Wet mat is the most commonly used.	9/12/2019 4:47 AM
2	Steam or radiant heat. (Typically, radiant heat is used.)	8/26/2019 11:33 AM
3	All of the following methods are allowed by our specs: 1) covering with wet burlap or canvas 2) continuous water mist 3) airtight seal using plastic curing blankets 4) steam curing	8/12/2019 11:49 AM
4	Steam Curing, Water Curing, and Radiant Heat Curing are all options given to the precasters.	8/7/2019 7:47 AM
5	Please refer to ODOT C&MS 515.15 for specific curing requirements at the following link: http://www.dot.state.oh.us/Divisions/ConstructionMgt/OnlineDocs/Specifications/2019CMS/2019_CMS_07192019_for_web_letter_size.pdf	8/6/2019 9:27 AM
6	Steam cure "or cure by other methods identified by the contractor's fabrication quality control plan". Curing compound is not allowed.	8/6/2019 5:16 AM
7	Moist curing is specified with Contractor option of steam curing with adherence to requirements.	8/5/2019 12:15 PM
8	steam cure in the winter and moist cure in the summer	8/1/2019 1:09 PM
9	Steam and Moist curing are allowed	8/1/2019 4:09 AM
10	steam or wet method with thermocouples "match cure"	7/31/2019 1:55 PM

Q12 Which curing technique is more prevalent in your state?

Answered: 27 Skipped: 16

#	RESPONSES	DATE
1	Moist or water curing (wet mat or soaker hoses)	9/12/2019 4:47 AM
2	Radiant heat	8/26/2019 11:34 AM
3	Moist curing with curing compound and burlap sheeting on the top flange.	8/21/2019 7:19 AM
4	Steam curing	8/12/2019 11:50 AM
5	Fabricators have the option of steam or saturated cover curing. Pub 408, Section 1107.03(d)5.g provides PennDOT's requirements on initial and intermediate curing phases. Each phase has temperature requirements. Typically, PennDOT utilizes the saturated covering in conjunction with a steam cure.	8/12/2019 6:44 AM
6	Steam Cure	8/9/2019 9:22 AM
7	Steam Curing	8/7/2019 7:47 AM
8	Steam curing	8/7/2019 3:33 AM
9	Steam curing.	8/6/2019 6:37 AM
10	Moist curing	8/6/2019 5:16 AM
11	Moist curing	8/5/2019 12:16 PM
12	Steam Curing	8/2/2019 9:29 AM
13	steam accelerated method	8/2/2019 9:09 AM
14	moist curing	8/2/2019 7:50 AM
15	Moist curing.	8/2/2019 4:50 AM
16	moist curing method	8/1/2019 3:34 PM
17	moist curing	8/1/2019 1:10 PM
18	radiant heat curing/electric	8/1/2019 6:49 AM
19	Moist (Wet) Curing and accelerated curing using low pressure steam or radiant heat curing.	8/1/2019 4:10 AM
20	Saturated cover curing.	7/31/2019 1:54 PM
21	Steam cured.	7/31/2019 1:10 PM
22	Steam	7/31/2019 12:48 PM
23	Moist curing.	7/31/2019 12:25 PM
24	Unsure	7/31/2019 11:54 AM
25	Steam methods are used by the Alaska fabricators Heated forms are used by the Washington fabricator	7/31/2019 11:44 AM
26	Steam	7/31/2019 11:42 AM
27	Wet cure	7/31/2019 11:40 AM

Q13 What method do you use to measure camber?

Answered: 25 Skipped: 18

#	RESPONSES	DATE
1	No prescribed method. Camber is only measured when there is a problem.	9/12/2019 4:49 AM
2	Deviation from stringline	8/26/2019 11:42 AM
3	At fabricator yard before shipping, measurements may be done using stringline or survey instruments. After beam erection, measurement is done using survey instruments.	8/12/2019 12:13 PM
4	a. Beams less than 70' – string line b. Longer than 70' – Surveyor's Transit	8/12/2019 6:45 AM
5	Survey, note it is up to the fabricator's inspector to submit camber measurements.	8/9/2019 9:36 AM
6	Inspectors manually measure the camber at the precast plant.	8/7/2019 7:48 AM
7	It is specified for the fabricator to manage camber if they think that the actual camber will be higher than the calculated camber. Typically the fabricator will take shots on the top of the beam at the ends and midspan and record the information.	8/7/2019 3:59 AM
8	Taut stringline with beams sitting on proper donnage.	8/6/2019 9:37 AM
9	Tape measure at midspan	8/6/2019 5:21 AM
10	None specifically prescribed in Specifications. Typically string line or shooting elevations.	8/5/2019 12:18 PM
11	survey	8/2/2019 9:14 AM
12	string line method	8/2/2019 7:53 AM
13	Tension line with measured offsets at the tenth points.	8/2/2019 5:05 AM
14	Survey shots at tenth points along the beam top flange.	8/1/2019 3:50 PM
15	Laser beam	8/1/2019 6:52 AM
16	Survey	8/1/2019 4:40 AM
17	None unless there is a concern. If needed, would use tape measure.	8/1/2019 2:30 AM
18	Survey or string line method.	7/31/2019 2:00 PM
19	Shots on top of beam after beam is set on substructures.	7/31/2019 1:13 PM
20	Not specified	7/31/2019 12:49 PM
21	Not specified.	7/31/2019 12:29 PM
22	Camber measured by fabricator	7/31/2019 12:13 PM
23	Camber measurement procedure listed in IM 570 at https://iowadot.gov/erl/current/IM/content/570.htm Measure elevations at beam ends and center after lifting and resetting beams within 3 hours of detensioning.	7/31/2019 12:02 PM
24	As I understand, chord offset is measured using a laser level. Girder lift off distance at mid-span (distance between bottom pan and concrete girder after release) is also measured for each girder	7/31/2019 11:53 AM
25	Survey	7/31/2019 11:42 AM

Q14 What tools do you use to measure camber?

Answered: 26 Skipped: 17

#	RESPONSES	DATE
1	Steel wire line	9/12/2019 4:49 AM
2	Stringline and tape measure	8/26/2019 11:42 AM
3	Stringline and survey.	8/21/2019 7:22 AM
4	See answer to previous question.	8/12/2019 12:13 PM
5	a. String lines b. Steel tape measure c. Surveyor's transit	8/12/2019 6:45 AM
6	Total Station	8/9/2019 9:36 AM
7	String Line and tape measure	8/7/2019 7:48 AM
8	See above	8/7/2019 3:59 AM
9	String, tape measure, 3 individuals.	8/6/2019 9:37 AM
10	Tape measure	8/6/2019 5:21 AM
11	None specifically prescribed in Specifications.	8/5/2019 12:18 PM
12	level	8/2/2019 9:14 AM
13	tape measure and string line	8/2/2019 7:53 AM
14	See 12.	8/2/2019 5:05 AM
15	Survey shots at tenth points along the beam top flange.	8/1/2019 3:50 PM
16	laser	8/1/2019 6:52 AM
17	Laser Level	8/1/2019 4:40 AM
18	tape measure	8/1/2019 2:30 AM
19	Survey equipment or string line and tape measure.	7/31/2019 2:00 PM
20	Survey Level.	7/31/2019 1:13 PM
21	Not specified	7/31/2019 12:49 PM
22	Not specified.	7/31/2019 12:29 PM
23	survey equipment (distance from tensioned strings are not used anymore)	7/31/2019 12:13 PM
24	Typically surveyed.	7/31/2019 12:02 PM
25	Laser level for chord-offset method Tape measure for girder lift-off	7/31/2019 11:53 AM
26	Depends on contractor - Level	7/31/2019 11:42 AM

Q15 How frequently do you measure camber?

Answered: 27 Skipped: 16

#	RESPONSES	DATE
1	Rarely. Only when there is a problem.	9/12/2019 4:49 AM
2	Once a week	8/26/2019 11:42 AM
3	Monthly measurement is required per our Specification.	8/21/2019 7:22 AM
4	Initial camber is measured before removal from the casting bed. During storage, camber is measured at frequencies < 60 days and within 7 to 21 days of shipment. It is also measured after beam erection when setting grades for the deck pour.	8/12/2019 12:13 PM
5	a. Camber is measured after cutting the strands (form release) or at beginning of beam storage b. According to Publication 408, Section 1107, camber verification is required within 2 weeks of shipping	8/12/2019 6:45 AM
6	At release and prior to shipment	8/9/2019 9:36 AM
7	Camber is measured when member is removed from the casting bed and at the time of final inspection.	8/7/2019 7:48 AM
8	It is specified at regular intervals and in the fabricator's camber control procedure. Some fabricators will check it every three days to control camber growth.	8/7/2019 3:59 AM
9	Post pour after release, prior to storage and prior to shipment. The contractor can request a measurement at any time.	8/6/2019 9:37 AM
10	Once	8/6/2019 5:21 AM
11	Once members have been placed on temporary supports for storage at release, camber measurements are taken and at 2 week intervals thereafter up to 120 days after detensioning.	8/5/2019 12:18 PM
12	we require camber measure on every project before shipping	8/2/2019 9:14 AM
13	once unless delayed construction	8/2/2019 7:53 AM
14	Only at the plant (acceptance) before shipment.	8/2/2019 5:05 AM
15	First within 72 hours of release. Additional times if it is out of tolerance. Then again when the beams are erected / before deck stay-in-place formwork is installed.	8/1/2019 3:50 PM
16	As needed	8/1/2019 6:52 AM
17	During construction	8/1/2019 4:40 AM
18	Only when there are problems	8/1/2019 2:30 AM
19	During every hands-on bridge safety inspection (every six years).	7/31/2019 2:00 PM
20	3 times, initial , 28 days and 21 days prior to installation	7/31/2019 1:57 PM
21	Only when beams are set.	7/31/2019 1:13 PM
22	Not specified	7/31/2019 12:49 PM
23	Every 2 weeks.	7/31/2019 12:29 PM
24	weekly	7/31/2019 12:13 PM
25	Once	7/31/2019 12:02 PM
26	Usually twice - immediately after release of prestress force and prior to transport	7/31/2019 11:53 AM
27	Contractor measures before deck placement.	7/31/2019 11:42 AM

Q16 At what time of the day do you measure camber?

Answered: 25 Skipped: 18

#	RESPONSES	DATE
1	Any day light hours.	9/12/2019 4:49 AM
2	No specific time	8/26/2019 11:42 AM
3	Not specified in our Specs.	8/21/2019 7:22 AM
4	Our spec says to measure at a time when camber and alignment of beam is not influenced by temporary differences in surface temperature.	8/12/2019 12:13 PM
5	Varies, while PennDOT does not have a specification limit on this, camber is typically checked in the early morning to avoid direct sunlight	8/12/2019 6:45 AM
6	No specified time.	8/9/2019 9:36 AM
7	Various	8/7/2019 7:48 AM
8	It is in the fabricator's camber control procedure. Some fabricators check it in the morning.	8/7/2019 3:59 AM
9	Not specified.	8/6/2019 9:37 AM
10	Unknown	8/6/2019 5:21 AM
11	These measurements are recorded in the morning to reduce the effects of solar radiation for each unit and entries include the date, time, weather conditions and measurements taken.	8/5/2019 12:18 PM
12	no requirements	8/2/2019 7:53 AM
13	There is no particular time of the day that we insist the camber be measured.	8/2/2019 5:05 AM
14	Various.	8/1/2019 3:50 PM
15	Mid day	8/1/2019 6:52 AM
16	Day time	8/1/2019 4:40 AM
17	N/A	8/1/2019 2:30 AM
18	Normally during midday.	7/31/2019 2:00 PM
19	Any time.	7/31/2019 1:13 PM
20	Not specified	7/31/2019 12:49 PM
21	Not specified.	7/31/2019 12:29 PM
22	Early in the morning	7/31/2019 12:13 PM
23	Not specified.	7/31/2019 12:02 PM
24	16-hours and about within about 1 or 2 days after setting on temporary bunking points	7/31/2019 11:53 AM
25	At discretion of contractor	7/31/2019 11:42 AM

Q17 What procedure or approach (such as pre-loading) do you use to minimize or control camber overgrowth?

Answered: 27 Skipped: 16

#	RESPONSES	DATE
1	No prescribed method.	9/12/2019 4:49 AM
2	Static weight (pre-loading) or change dunnage position	8/26/2019 11:42 AM
3	Pre-loading or ballasting.	8/21/2019 7:22 AM
4	Our spec assumes we can reasonably predict the camber for beams that will be erected and have the deck formed between 30 and 180 days of prestress release. For beams between 30 and 180 days old, fabricator must report if cambers become greater than 1 inch plus or minus beyond the erection camber given in the bridge plan. Limits are specified on beam end overhang during storage. For beams expected to be > 180 days old at erection, contractor is required to submit a plan to prevent/accommodate excess camber.	8/12/2019 12:13 PM
5	a. Only done on occasions. Pre-loading is accomplished by loading with precast roadway barrier or similar. In general, camber is not an issue.	8/12/2019 6:45 AM
6	Precast barrier has been used to pre-load, but results have not been successful.	8/9/2019 9:36 AM
7	Pre-loading is occasionally used on shorter beams.	8/7/2019 7:48 AM
8	Preloading is used by the fabricator to control camber growth.	8/7/2019 3:59 AM
9	Not permitted.	8/6/2019 9:37 AM
10	Pre-loading very rarely done, except if long term storage.	8/6/2019 5:21 AM
11	The Contractor is required to submit a camber management plan prior to fabrication indicating the method for controlling camber. Pre-loading is the primary method utilized. The Contractor is required to implement the plan if specific thresholds are exceeded. Contractors are required to implement the camber management plan for any members that will be erected 120 days or more after detensioning. Where a change in construction schedule occurs which will result in erection 120 days or more after detensioning, the Contractor is required to implement the camber management plan. The Contractor may submit a request to delay the implementation of the camber management plan to the next scheduled camber measurement if documentation is submitted showing the camber growth is following an established camber development path that will not exceed the camber tolerance at erection under the current construction schedule.	8/5/2019 12:18 PM
12	We don't do this	8/2/2019 9:14 AM
13	have had to preload in construction in a few cases	8/2/2019 7:53 AM
14	We have used pre-loading on occasion.	8/2/2019 5:05 AM
15	none	8/1/2019 3:50 PM
16	Preloading	8/1/2019 6:52 AM
17	None	8/1/2019 4:40 AM
18	none	8/1/2019 2:30 AM
19	None.	7/31/2019 2:00 PM
20	nonea	7/31/2019 1:57 PM
21	We have used pre-load or temporary prestressing.	7/31/2019 1:13 PM
22	Pre-loading	7/31/2019 12:49 PM
23	Up to the contractor/fabricator.	7/31/2019 12:29 PM
24	Over growth has not been a concern. Specifications allow fabricator to move the supports in the storage to increase camber, if needed.	7/31/2019 12:13 PM
25	Fabricator may adjust supports in yard to control camber growth.	7/31/2019 12:02 PM
26	Fabricators can sag (parabolic vertical depression) for Alaska style decked bulb-T girders (most commonly used in out state)	7/31/2019 11:53 AM
27	Pre-loading has occasionally been used in our state.	7/31/2019 11:42 AM

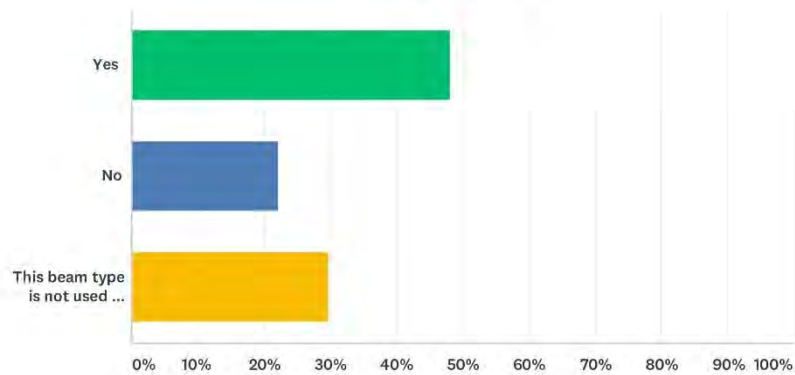
Q18 How do you typically deal with inaccurate predictions of camber?

Answered: 27 Skipped: 16

#	RESPONSES	DATE
1	Fabricator - TxDOT specifications state, "Variations greater than those specified in Table 3 (there is no camber tolerance) are subject to review. However, these tolerances do not relieve the Contractor from the responsibility of furnishing a completed structure that is in reasonably close conformity with the lines, grades, cross-sections, dimensions, and details specified. Correct members not meeting these tolerances at no additional expense to the Department, to achieve a satisfactory completed structure. This also includes costs for correction due to variations in vertical beam camber. Correction may require replacement of the member. In the Field - For girder with significantly less than predicted camber tend to be rejected. For girders with higher than predicted camber, we will try to adjust the roadway grade. If we cannot, then in rare cases, these are rejected.	9/12/2019 4:49 AM
2	Adjustment to deck finished grade profile when necessary	8/26/2019 11:42 AM
3	Accommodate the product into the bridge by pedestal adjustments, build-up adjustments.	8/21/2019 7:22 AM
4	For cambers that are low, additional stool/haunch above the beams is built in to get the deck to proper grade. For cambers that are high, deck grades are adjusted (raised) in the field to accommodate cambers and avoid negative stools/haunches.	8/12/2019 12:13 PM
5	Case by case, limited cases. Measure deflections at erection, deck placement and barrier placement. The deflections are assessed to ensure water does not pond on the bridge.	8/12/2019 6:45 AM
6	Increase haunch thickness to allow camber variability. Place shims to adjust bearing seat elevations.	8/9/2019 9:36 AM
7	The bridge deck buildup (haunch) accommodates minor inaccuracies in camber predictions.	8/7/2019 7:48 AM
8	If camber is predicted inaccurately it is sometimes necessary to adjust the deck overpour to account for it.	8/7/2019 3:59 AM
9	ODOT C&MS 511.07 at the following link: http://www.dot.state.oh.us/Divisions/ConstructionMgt/OnlineDocs/Specifications/2019CMS/2019_CMS_07192019_for_web_letter_size.pdf	8/6/2019 9:37 AM
10	May have to add "hat" bars (supplemental bars to ensure stirrups project 3" min. into the deck).	8/6/2019 5:21 AM
11	Preloading and Contractor adjustments to seat elevations or grade if approved by the Department. Use of top strands has been proposed to alleviate continued inaccuracies and reduce camber growth.	8/5/2019 12:18 PM
12	we use 1.5 in of fillet on our NU girder and we add hat bars in the field	8/2/2019 9:14 AM
13	make up for it in the haunch	8/2/2019 7:53 AM
14	As with steel beams, the camber for prestress concrete beams is about twice that anticipated at installation. Over time, with creep and with the camber deflections decreasing, there are usually other more significant issues which will warrant bridge rehabilitation or replacement.	8/2/2019 5:05 AM
15	We add a 1" camber tolerance into the required haunch thickness for all spans greater than 80'. This avoids camber issues in most cases. In long-span bulb tee girders we deal with issues such as accepting net negative camber in the final condition in approximately 25% of those bridges (spans > 120-feet).	8/1/2019 3:50 PM
16	Slab Haunch - "A" Dimension	8/1/2019 6:52 AM
17	Adjust screeds,	8/1/2019 4:40 AM
18	A minimum coping thickness is applied on top of the beam that provides room for errors in predicted camber.	8/1/2019 2:30 AM
19	Asphalt overlay.	7/31/2019 2:00 PM
20	modify haunch depth if necessary	7/31/2019 1:57 PM
21	Adjust the riser or change the profile on the deck.	7/31/2019 1:13 PM
22	Case-by-case	7/31/2019 12:49 PM
23	Adjust bridge seats, bolster/haunch and grade.	7/31/2019 12:29 PM
24	The contractor is required to construct the bearing seat elevation based on actual measured camber.	7/31/2019 12:13 PM
25	Built-in tolerance in stirrups, then modify road profile, then stirrup height modification.	7/31/2019 12:02 PM
26	If differential camber exceeds contract tolerances (i.e., 0.25" at ends and 0.5" at midspan) then forces devices cast in the girders are used to level up girders	7/31/2019 11:53 AM
27	Contractor has to adjust deck at his expense.	7/31/2019 11:42 AM

Q19 Are box beams prone to inaccurate camber predictions?

Answered: 27 Skipped: 16



ANSWER CHOICES

RESPONSES

Yes	48.15%	13
No	22.22%	6
This beam type is not used in my state	29.63%	8
TOTAL		27

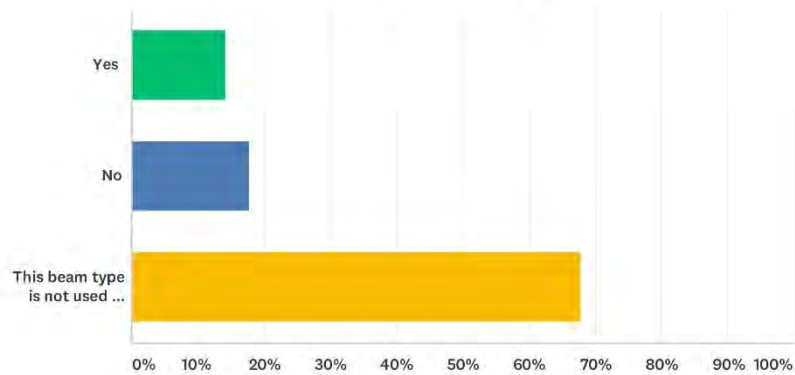
Q20 Is there a specific size(s) of the member that is more prone?

Answered: 14 Skipped: 29

#	RESPONSES	DATE
1	no	9/12/2019 4:49 AM
2	48 inch	8/12/2019 6:45 AM
3	No, based on internal research camber variation is the same over different span/depth ratios. Note, girders with larger cambers will have larger variation, but the over/under camber percentage is the same.	8/9/2019 9:41 AM
4	Box Beam camber predictions have significantly improved since the implementation of NCDOT's camber research.	8/7/2019 7:49 AM
5	No.	8/7/2019 3:59 AM
6	No	8/6/2019 9:37 AM
7	Box beams designed for concrete strength in upper range of allowable with larger number of strands are more prone, but can typically be dealt with by Contractor field adjustments.	8/5/2019 12:18 PM
8	We have not investigated this issue.	8/2/2019 5:06 AM
9	no	8/1/2019 1:22 PM
10	No.	7/31/2019 2:00 PM
11	No	7/31/2019 1:13 PM
12	No	7/31/2019 12:49 PM
13	No.	7/31/2019 12:30 PM
14	Not sure	7/31/2019 11:43 AM

Q21 Are cored slabs prone to inaccurate camber predictions?

Answered: 28 Skipped: 15



ANSWER CHOICES	RESPONSES	
Yes	14.29%	4
No	17.86%	5
This beam type is not used in my state	67.86%	19
TOTAL		28

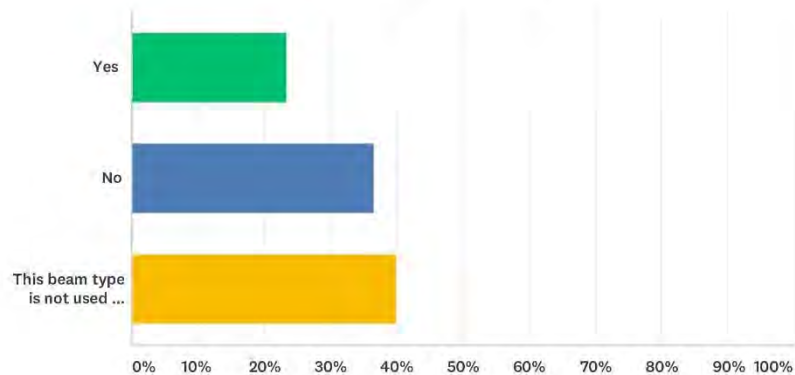
Q22 Are there specific cored slab size(s) of the member that is more prone?

Answered: 4 Skipped: 39

#	RESPONSES	DATE
1	Cored Slab camber predictions have significantly improved since the implementation of NCDOT's camber research.	8/7/2019 7:50 AM
2	No.	8/7/2019 3:59 AM
3	Voided slabs designed for concrete strength in upper range of allowable with larger number of strands are more prone, but can typically be dealt with by Contractor field adjustments (assume voided slabs are similar to cored slabs in response here and in Question 20).	8/5/2019 12:19 PM
4	No	7/31/2019 12:50 PM

Q23 Are AASHTO I-beams prone to inaccurate camber predictions?

Answered: 30 Skipped: 13



ANSWER CHOICES	RESPONSES	
Yes	23.33%	7
No	36.67%	11
This beam type is not used in my state	40.00%	12
TOTAL		30

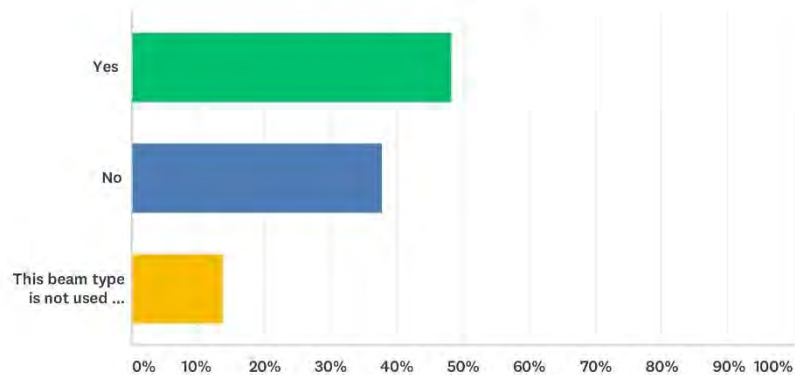
Q24 Are there specific I-beam size(s) of the member that is more prone?

Answered: 7 Skipped: 36

#	RESPONSES	DATE
1	Larger sizes that approach maximum span lengths for that size	8/26/2019 2:18 PM
2	No	8/6/2019 9:38 AM
3	These beam types are only used on in-kind widening and rarely utilized. Historically, VDOT previously designed using 5 ksi concrete and cracking overload so less strain differential. Type 6 are more prone where span lengths are near maximum range for section or where shallower section than existing is utilized due to existing vertical clearances and exterior widening.	8/5/2019 12:20 PM
4	We do not know.	8/2/2019 5:07 AM
5	All	8/1/2019 4:40 AM
6	no	7/31/2019 1:14 PM
7	Not sure	7/31/2019 11:43 AM

Q25 Are bulb T beams prone to inaccurate camber predictions?

Answered: 29 Skipped: 14



ANSWER CHOICES	RESPONSES	
Yes	48.28%	14
No	37.93%	11
This beam type is not used in my state	13.79%	4
TOTAL		29

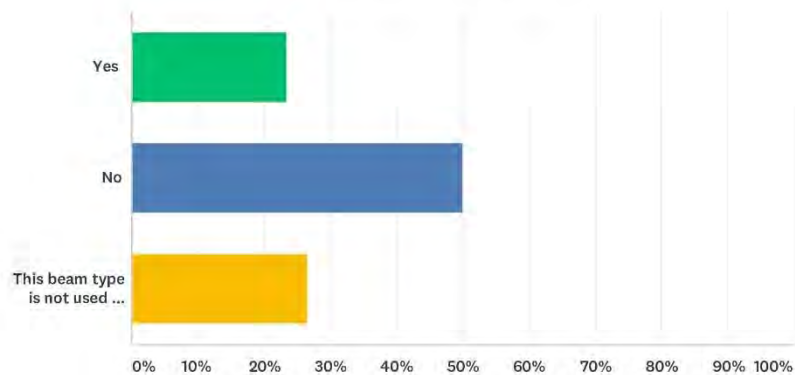
Q26 Are there specific bulb T beams size(s) of the member that is more prone?

Answered: 14 Skipped: 29

#	RESPONSES	DATE
1	no	9/12/2019 4:49 AM
2	Larger sizes that approach maximum span lengths for that size	8/26/2019 2:20 PM
3	No, see previous response.	8/9/2019 9:43 AM
4	No.	8/7/2019 4:00 AM
5	No, 36" to 72". To clarify #21, we haven't had some camber problems, but not significant lately.	8/6/2019 5:23 AM
6	Whether 29" bulb-T sections (smallest size used by VDOT) are more prone where designed for concrete strength in upper range of allowable with larger number of strands are more prone to experience inaccurate camber predictions is debatable but appears to depend on whether design-bid-build or design-build procurement process is used. 85" bulb-T and primarily 93" bulb-T sections are more prone where designed for concrete strength in upper range of allowable with larger number of strands where span lengths are near maximum range for section. Stability during fabrication, transportation and erection are also issues with these sections.	8/5/2019 12:20 PM
7	We do not know.	8/2/2019 5:08 AM
8	Generally any bulb tee with span lengths greater than 125-feet.	8/1/2019 3:52 PM
9	All	8/1/2019 4:40 AM
10	No	7/31/2019 12:50 PM
11	No.	7/31/2019 12:30 PM
12	The cambers typically have been lower than predicted. The longer beams would tend to be more troublesome since the cambers are larger.	7/31/2019 12:04 PM
13	Although no systematic evaluation has been undertaken by the Department, it seems that deeper, longer girders are more prone to camber variability	7/31/2019 11:55 AM
14	Not Sure	7/31/2019 11:44 AM

Q27 Are other beam types prone to inaccurate camber predictions?

Answered: 30 Skipped: 13



ANSWER CHOICES	RESPONSES	
Yes	23.33%	7
No	50.00%	15
This beam type is not used in my state	26.67%	8
TOTAL		30

Q28 Are there specific other beams size(s) of the member that is more prone?

Answered: 6 Skipped: 37

#	RESPONSES	DATE
1	All beam types can be subject to inaccurate camber issues. We use bulb T beams, box beams, solid slab beams, and U beams. Rarely we see the longer beams with higher amounts of strand have more inaccurate camber issues. Typically, inaccurate camber issues arise in the different materials and practices the various precast fabricators use.	9/12/2019 4:50 AM
2	We use an Inverted T section 300mm to 800mm It seems they are more prone	8/2/2019 9:17 AM
3	We do not know.	8/2/2019 5:08 AM
4	no	7/31/2019 1:14 PM
5	Not sure	7/31/2019 11:44 AM
6	no	7/31/2019 11:25 AM

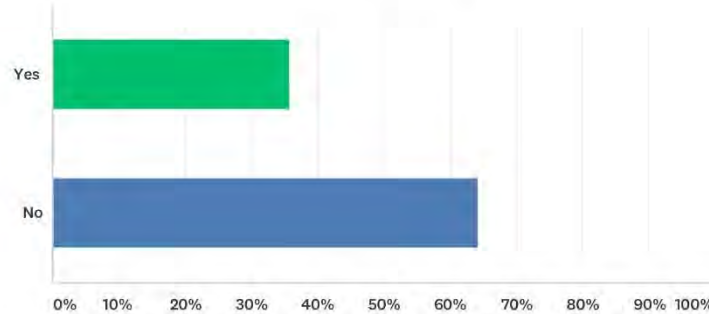
Q29 On past deck replacement projects MDOT has observed that existing prestressed and steel beams/girders do not rebound to the original camber when the deck weight is removed. Do you have a procedure or tool that you use to predict beam rebound in deck replacement projects?

Answered: 28 Skipped: 15

#	RESPONSES	DATE
1	no	9/12/2019 4:50 AM
2	No.	8/26/2019 2:23 PM
3	N/A	8/21/2019 7:23 AM
4	No	8/12/2019 12:14 PM
5	No	8/12/2019 6:46 AM
6	No.	8/9/2019 9:44 AM
7	No procedure or tool, but beam rebound should be considered in deck replacement projects.	8/7/2019 7:50 AM
8	No.	8/7/2019 4:00 AM
9	Please see BDM Section 403.5 at the following link: http://www.dot.state.oh.us/Divisions/Engineering/Structures/standard/Bridges/BDM/2019_BDM_07-19-19.pdf	8/6/2019 9:42 AM
10	At times we've measured girder location before and after deck removal, but seem to have gotten away from that practice.	8/6/2019 5:24 AM
11	No.	8/5/2019 12:21 PM
12	We had similar situation on few AASHTO type girders. We don't have a tool to predict beam rebound if any	8/2/2019 9:18 AM
13	make it up in the haunch	8/2/2019 7:55 AM
14	No. We do, however, believe that although camber predictions may be inaccurate, they are at least consistently off and conservative. We measure camber before placing the new deck -- the camber is still well within the predicted limits.	8/2/2019 5:14 AM
15	No. SCDOT has performed less than five total deck replacements.	8/1/2019 3:53 PM
16	No	8/1/2019 1:24 PM
17	No	8/1/2019 6:53 AM
18	No	8/1/2019 4:42 AM
19	No	8/1/2019 2:31 AM
20	no.	7/31/2019 2:02 PM
21	No	7/31/2019 1:58 PM
22	No	7/31/2019 1:15 PM
23	No	7/31/2019 12:50 PM
24	No.	7/31/2019 12:31 PM
25	We do not.	7/31/2019 12:14 PM
26	No. I don't know that we've done very many deck replacements on prestressed beams.	7/31/2019 12:05 PM
27	No	7/31/2019 11:56 AM
28	No	7/31/2019 11:45 AM

Q30 Have you measured the rebound of existing beams/girders after the deck weight is removed and deflection of the beams/girders after the new deck has been constructed on deck replacement projects?

Answered: 28 Skipped: 15



ANSWER CHOICES	RESPONSES	
Yes	35.71%	10
No	64.29%	18
TOTAL		28

Q31 Have you found a correlation between the rebound and the deflection?

Answered: 10 Skipped: 33

#	RESPONSES	DATE
1	No	8/12/2019 6:46 AM
2	Yes	8/6/2019 9:42 AM
3	Some	8/6/2019 5:25 AM
4	Yes but inconclusive because of the creep and strength gain effect	8/2/2019 9:20 AM
5	No	8/2/2019 7:55 AM
6	As you stated previously in this questionnaire, most states do not see a significant rebound. The same is true of our state.	8/2/2019 5:17 AM
7	No	8/1/2019 4:43 AM
8	No.	7/31/2019 2:02 PM
9	No	7/31/2019 12:51 PM
10	Yes.	7/31/2019 12:31 PM

APPENDIX C: Creep and Shrinkage Models

C.1. Creep Coefficient Prediction Models

Over the past 30 years, several models have been proposed for prediction of creep coefficient (ACI 209.2R-08). In this thesis, there are seven different creep models have been employed: AASHTO LRFD 2020, ACI 209R-92, Bažant-Baweja B3, CEB MC90, CEB MC90-99, GL 2000 and fib MC 2010. Out of these, AASHTO LRFD 2020 is divided into two separate creep models: AASHTO Body 2020 and AASHTO Commentary 2020. Therefore, in total eight different creep models are investigated in this thesis.

C.1.1. AASHTO LRFD 2020

The model proposed by AASHTO should be used for calculating prestressing loss in bridges other than segmentally constructed ones (AASHTO LRFD 2020). Furthermore, code allows it to use also for determining the effects of creep in deflection calculations. There are four different factors which are needed to be considered while calculating the creep coefficient.

$$\psi(t, t_1) = 1.9k_s k_{hc} k_f k_{td} t_i^{-0.118} \quad \text{C-1}$$

$$k_{sb} = 1.45 - 0.13 \left(\frac{V}{S} \right) \geq 1.0 \quad \text{C-2}$$

$$k_{sc} = \left[\frac{\frac{t}{26e^{0.36(V/S)} + t}}{\frac{t}{45 + t}} \right] \left[\frac{1.80 + 1.77e^{-0.54(V/S)}}{2.587} \right] \quad \text{C-3}$$

$$k_{hc} = 1.56 - 0.008H \quad \text{C-4}$$

$$k_f = \frac{5}{1 + f'_{ci}} \quad \text{C-5}$$

$$k_{td} = \frac{t}{12 \left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20} \right) + t} \quad \text{C-6}$$

Where:

$\psi(t, t_1)$ is the creep coefficient at concrete at t due to load applied at t_i ,

H is the average annual ambient relative humidity (%),

k_{sb}/k_{sc} is factor for the effect of the volume-to-surface ratio of the component,

k_{hc} is humidity factor for creep,

k_f is factor for the effect of concrete strength,

k_{td} is the time development factor,

t is the maturity of concrete (days),

V/S is the volume-to-surface ratio (in.),

t_i is the age of concrete at the time of load application (days),

f'_{ci} is the design concrete compressive strength at the time of load application (ksi).

As it can be seen above, there are two equations for the effect of volume-to-surface ratio: k_{sb} is for the Body provision of AASHTO LRFD 2020, and k_{sc} is for the Commentary provision of AASHTO LRFD 2020. The creep coefficients have been calculated by using these two approaches separately. According to AASHTO LRFD 2020, it is stated that body equation for volume-to-surface ratio, k_{sb} , is a simplified from the formula stated in the commentary part, k_{cb} .

Moreover, it is emphasized that the model is applicable for the design concrete compressive strengths up to 15 ksi. Furthermore, if the concrete age is not known at the time of loading, f'_{ci} may be taken as 80 % of 28 day concrete compressive strength, f'_c .

C.1.2. ACI 209R-92

This model is proposed by Branson and Christiason (1971), and with slight modification, ACI Committee introduced it in ACI 209R-82 in 1982. Further, the developed model was incorporated by the same committee in 1992 in ACI 209R-92 (ACI 209.2R-08). The approach of this model is to estimate the ultimate creep coefficient, ϕ_u , by considering 6 different parameters such as concrete mixture properties, member geometry, and loading conditions. Then, a time-rate function, the first part of the Eqn. C-7 below, is utilized to predict the development of creep coefficient.

$$\phi(t, t_o) = \frac{(t - t_o)^\psi}{d + (t - t_o)^\psi} \phi_u \quad \text{C-7}$$

Where:

$\phi(t, t_o)$ is the creep coefficient at concrete age t due to a load applied at the age t_o ,

d (days) and ψ are constants for a given member shape,

ϕ_u is the ultimate creep coefficient.

Factors			Variables considered	Standard conditions
Concrete (creep and shrinkage)	Concrete composition	Cement paste content	Type of cement	Type I and III
		Water-cement ratio	Slump	70 mm (2.7 in.)
		Mixture proportions	Air content	≤ 6%
		Aggregate characteristics	Fine aggregate percentage	50%
		Degrees of compaction	Cement content	279 to 446 kg/m ³ (470 to 752 lb/yd ³)
	Initial curing	Length of initial curing	Moist cured	7 days
			Steam cured	1 to 3 days
		Curing temperature	Moist cured	23.2 ± 2 °C (73.4 ± 4 °F)
			Steam cured	≤ 100 °C (≤ 212 °F)
		Curing humidity	Relative humidity	≥ 95%
Member geometry and environment (creep and shrinkage)	Environment	Concrete temperature	Concrete temperature	23.2 ± 2 °C (73.4 ± 4 °F)
		Concrete water content	Ambient relative humidity	40%
	Geometry	Size and shape	Volume-surface ratio or minimum thickness	V/S = 38 mm (1.5 in.)
				150 mm (6 in.)
Loading (creep only)	Loading history	Concrete age at load application	Moist cured	7 days
			Steam cured	1 to 3 days
		During of loading period	Sustained load	Sustained load
		Duration of unloading period	—	—
	Stress conditions	Number of load cycles	—	—
		Type of stress and distribution across the section	Compressive stress	Axial compression
		Stress/strength ratio	Stress/strength ratio	≤ 0.50

Figure C.1. Factors affecting concrete creep and shrinkage (ACI 209.2R-08)

For the time ratio in Eqn. C-7, ACI 209R-92 recommends an average value of 10 and 0.6 for d and ψ , respectively. On the other hand, the shape and size effect could be considered if ψ and d are replaced by 1.0 and f , where f is parameter for considering the effect of size effect and defined in C-8.

$$f = 26e^{0.36(V/S)} \quad \text{C-8}$$

Where:

V/S is volume-to-surface ratio (in.).

Moreover, in case there is lack of information about specific creep data for local aggregates, and conditions, the average value proposed for ϕ_u is 2.35.

For conditions other than standard conditions, as shown in Figure C.1, the value of ϕ_u is modified by six factors.

$$\phi_u = 2.35\gamma_c \quad \text{C-9}$$

$$\gamma_c = \gamma_{c,to} \gamma_{c,RH} \gamma_{c,vs} \gamma_{c,s} \gamma_{c,\psi} \gamma_{c,\alpha} \quad \text{C-10}$$

Where:

γ_c is the cumulative product of the applicable correction factors,

$\gamma_{c,to}$ is the loading age correction factor,

$\gamma_{c,RH}$ is the relative humidity correction factor,

$\gamma_{c,vs}$ is the volume-to-surface correction factor,

$\gamma_{c,s}$ is the slump correction factor,

$\gamma_{c,\psi}$ is the fine aggregate correction factor,

$\gamma_{sh,\alpha}$ is the air content correction factor.

For ages at application of load greater than 7 days for moist-cured or 1 to 3 days for steam-cured concrete, the age of loading factor, $\gamma_{c,to}$, is estimated from Eqns. C-11 and C-12.

$$\text{For moist curing:} \quad \gamma_{c,to} = 1.25t_o^{-0.118} \quad \text{C-11}$$

$$\text{For steam curing:} \quad \gamma_{c,to} = 1.13t_o^{-0.094} \quad \text{C-12}$$

Where:

t_o is the age of concrete at loading (days).

The ambient relative humidity factor, $\gamma_{c,RH}$, is obtained from Eqns. C-13 and C-14.

$$\text{For } h \geq 0.40: \quad \gamma_{c,RH} = 1.27 - 0.67h \quad \text{C-13}$$

$$\text{For } h < 0.40: \quad \gamma_{c,RH} > 1.0 \quad \text{C-14}$$

Where:

h is relative humidity in decimals.

The model indicates that if h is lower than 0.40, value for $\gamma_{c,RH}$ higher than 1.0 should be used.

The size correction factor of the member, $\gamma_{c,vs}$ is obtained from Eqn. C-15.

$$\gamma_{c,vs} = \frac{2}{3} \left(1 + 1.13e^{-0.213\left(\frac{V}{S}\right)} \right) \quad \text{C-15}$$

Where:

V/S is volume-to-surface ratio (in.).

There is also alternative method presented in the report, however, since it was not utilized in the analysis, it was not mentioned in this thesis.

The correction factor for slump of concrete, $\gamma_{c,s}$, is obtained from Eqn. C-16.

$$\gamma_{c,s} = 0.82 + 0.067s \quad \text{C-16}$$

Where:

s is the slump of concrete (in.),

The correction factor fine aggregate, $\gamma_{c,\psi}$ is obtained from C-17.

$$\gamma_{c,\psi} = 0.88 + 0.0024\psi \quad \text{C-17}$$

Where:

ψ is the ratio of fine aggregate to total aggregate by weight (%).

The air content factor, $\gamma_{c,\alpha}$, is obtained from Eqn. C-18.

$$\gamma_{c,\alpha} = 0.46 + 0.09\alpha \geq 1 \quad \text{C-18}$$

It important to note that the equations above is applicable for normal-weight, sand light-weight and all light-weight concrete (using both moist and steam curing, and Types I and III cements) under standard conditions.

C.1.3. Bažant-Baweja B3

Bažant-Baweja B3 Model is developed in 1995 and further modified by Bažant and Baweja in 2000. It is stated that the main source of error is the effects of concrete composition and design strength on model parameters. There are some restrictions to be able to use this model as mentioned below.

- $0.35 \leq w/c \leq 0.85$
- $2.5 \leq a/c \leq 13.5$
- $2500 \text{ psi} \leq f_{cm28} \leq 10000 \text{ psi}$
- $270 \text{ lb}^3/\text{yd}^3 \leq c \leq 1215 \text{ lb}^3/\text{yd}^3$

Where:

f_{cm28} is the mean 28-day standard cylinder compression strength of concrete (psi),

w/c is the water-to-cement ratio by weight,

c is the cement content, (lb^3/yd^3),

a/c is the aggregate-to-cement ratio by weight.

Moreover, it is noted that this model is applicable if the stresses are service stresses i.e. about $0.45f_{cm28}$, and if the member is cured at least one day. It is suggested that if only the design

strength is known, then f_{cm28} may be assumed 1200 psi more than the specified concrete compressive strength.

Model provides two different creep coefficients: basic creep and drying creep. Basic creep is measured on specimens that are sealed to prevent the ingress or egress of moisture from or to its environment. On the other hand, drying creep is the strain remaining after subtracting shrinkage, elastic and basic creep strain from the total measured strain on nominally identical specimens in a drying environment (ACI 209.2R-08). Therefore, two sets of creep coefficient are determined: one with basic creep only; and the other one is drying creep only. While using the model in the analysis, total creep coefficient, the summation of basic and drying creep, is used.

According to this model, basic creep is composed of three terms: an ageing viscoelastic term, a nonageing viscoelastic term, and an ageing flow term as shown in the expression below (C-19), respectively.

$$C_o(t, t_o) = \frac{q_2 Q(t, t_o) + q_3 \ln[1 + (t - t_o)^n] + q_4 \ln(t/t_o)}{q_1} \quad \text{C-19}$$

$$q_1 = \frac{0.6}{E_{cm28}} \quad \text{C-20}$$

❖ Ageing viscoelastic term: $q_2 Q(t, t_o)$

$$q_2 = 86.814(10^{-6})c^{0.5}f_{cm28}^{-0.9} \quad \text{C-21}$$

$$Q(t, t_o) = Q_f(t_o) \left[1 + \left(\frac{Q_f(t_o)}{Z(t, t_o)} \right)^{r(t_o)} \right]^{-1/(t_o)} \quad \text{C-22}$$

$$Q_f(t_o) = [0.086(t_o)^{2/9} + 1.21(t_o)^{4/9}]^{-1} \quad \text{C-23}$$

$$Z(t, t_o) = (t_o)^{-m} \ln[1 + (t - t_o)^n] \quad \text{C-24}$$

$$r(t_o) = 1.7(t_o)^{0.12} + 8 \quad \text{C-25}$$

❖ Nonageing viscoelastic term: $q_3 \ln[1 + (t - t_o)^n]$

$$q_3 = 0.29(w/c)^4 q_2 \quad \text{C-26}$$

❖ Ageing flow term: $q_4 \ln(t/t_o)$

$$q_4 = 0.14(10^{-6})(a/c)^{-0.7} \quad \text{C-27}$$

On the other hand, the drying creep is defined as:

$$C_d(t, t_o, t_c) = \frac{q_5 [e^{-8H(t)} - e^{-8H(t_o)}]^{0.5}}{q_1} \quad \text{C-28}$$

$$q_5 = 0.757 f_{cm28}^{-1} |\varepsilon_{sh\infty}(10^6)|^{-0.6} \quad \text{C-29}$$

$$H(t) = 1 - (1 - h)S(t - t_c) \quad \text{C-30}$$

$$H(t_o) = 1 - (1 - h)S(t_o - t_c) \quad \text{C-31}$$

$$S(t - t_c) = \tanh \left[\left(\frac{t - t_c}{\tau_{sh}} \right)^{0.5} \right] \quad \text{C-32}$$

$$S(t_o - t_c) = \tanh \left[\left(\frac{t_o - t_c}{\tau_{sh}} \right)^{0.5} \right] \quad \text{C-33}$$

$$\tau_{sh} = 190.8 t_c^{-0.08} f_{cm28}^{-0.25} [2k_s (V/S)]^2 \quad \text{C-34}$$

Where:

t_o is the age of concrete loading (days),

t_c is the age drying began or end of moist curing (days),

m and n are empirical parameters, taken as 0.5 and 0.1 for normal concretes, respectively,

w/c is the water-to-cement ratio by weight,

h is the relative humidity in decimals,

c is the cement content (lb/yd³),

a/c is the aggregate-to-cement ratio by weight,

E_{cm28} is the mean concrete modulus of elasticity at 28 days (psi),

f_{cm28} is the mean concrete compressive strength at 28 days (psi),

$\varepsilon_{sh\infty}$ is the ultimate shrinkage strain (in./in.),

τ_{sh} is the shrinkage half-time (days),

V/S is the volume-to-surface ratio (in.),

k_s is the cross-section shape correction factor, may be taken as 1.0.

C.1.4. CEB MC90

This model has a similar concept that ACI 209R-92 model has such that both uses the ultimate creep coefficient with time-function. It was developed by Muller and Hilsdorf in 1990,

and was intended to predict time-dependent behavior of concrete members. Like in other prediction models, this model has also some application limits which are provided below:

- Average compressive strength at 28 days should be between 2900 psi and 13000 psi.
- Member should be moist cured at normal temperatures not longer than 14 days.
- Mean ambient relative humidity should not be lower than 40 %.
- Age at loading, t_o , should be at least 1 day.
- Sustained stress should not exceed 40 % of the mean concrete compressive strength, f_{cmto} at the time of loading, t_o .

$$\phi(t, t_o) = \phi_{28}(t, t_o) \frac{E_{cmto}}{E_{cm28}} \quad \text{C-35}$$

$$\phi_{28}(t, t_o) = \phi_o \beta_c(t, t_o) \quad \text{C-36}$$

$$\beta_c(t, t_o) = \left[\frac{(t - t_o)/t_1}{\beta_H + (t - t_o)/t_1} \right]^{0.3} \quad \text{C-37}$$

$$\beta_H = 150 \left[(1 + (1.2h/h_o)^{18}) \right] (V/S)/(V/S)_o + 250 \alpha_3 \leq 1500 \alpha_3 \quad \text{C-38}$$

$$\phi_o = \phi_{RH}(h) \beta(f_{cm28}) \beta(t_o) \quad \text{C-39}$$

$$\phi_{RH}(h) = \left[1 + \frac{1 - h/h_o}{\sqrt[3]{0.1(V/S)/(V/S)_o}} \alpha_1 \right] \alpha_2 \quad \text{C-40}$$

$$\beta(f_{cm28}) = \frac{5.3}{\sqrt{f_{cm28}/f_{cmo}}} \quad \text{C-41}$$

$$\beta(t_o) = \frac{1}{0.1 + (t_{o,adj}/t_1)^{0.2}} \quad \text{C-42}$$

$$t_{o,adj} = t_o \left[\left(\frac{9}{2 + t_o^{1.2}} \right) + 1 \right]^\alpha \quad \text{C-43}$$

$$\alpha_1 = 1 \quad \text{C-44}$$

$$\alpha_2 = 1 \quad \text{C-45}$$

$$\alpha_3 = 1 \quad \text{C-46}$$

Where:

$\phi(t, t_o)$ is the creep coefficient function,

$\phi_{28}(t, t_o)$ is the 28-day creep coefficient,

ϕ_o is the notional creep coefficient,

$\beta_c(t, t_o)$ is the coefficient describing the development of creep with time,

f_{cm28} is the mean compressive strength of concrete at the age of 28 days (psi),

f_{cmo} is taken as 1450 psi,

E_{cm28} is the mean modulus of elasticity of concrete at the age of 28 days (psi),

E_{cmto} is mean the modulus of elasticity at the time of loading, t_o , (psi),

h is the relative humidity in decimals,

h_o is taken as 1.0,

V/S is the volume-to-surface ratio (in.),

$(V/S)_o$ is taken as 2 in.,

α is a power that depends on the type of cement (Table C.1),

α_1 , α_2 , and α_3 are coefficients that depend on mean compressive strength of concrete,

t_1 is taken as 1 day,

t is the age of concrete (days),

t_o is the age of concrete at loading (days),

$t_{o,adj}$ is the age of concrete at loading adjusted for cement type only (days).

Table C.1. Parameter α

Type of Cement	α
Slowly hardening cement	-1
normal or rapid hardening cement	0
rapid hardening high-strength cement	1

The effect of temperature on the creep coefficient may be taken into account by modifying the age at loading t_o , however, since it was not considered in the analysis, details about it was not provided in this thesis.

C.1.5. CEB MC90-99

The CEB MC90-99 includes the latest improvements to the CEB MC90 model and is very similar to it, however it was adjusted to take the particular characteristics of high-strength concretes into account. Like in CEB MC90 model, this model has the same restrictions but the range for average concrete compressive strength at 28 days was specified as 2175 psi and 17400 psi.

Moreover, the same formulas stated for CEB MC90 model are valid for this method with the exception that correction factors, α_1 , α_2 and α_3 in Eqns. between C-44 and C-46, should be replaced as follows:

$$\alpha_1 = \left(\frac{3.5f_{cmo}}{f_{cm28}} \right)^{0.7} \quad \text{C-47}$$

$$\alpha_2 = \left(\frac{3.5f_{cmo}}{f_{cm28}} \right)^{0.2} \quad \text{C-48}$$

$$\alpha_3 = \left(\frac{3.5f_{cmo}}{f_{cm28}} \right)^{0.5} \quad \text{C-49}$$

Where:

f_{cm28} is the mean compressive strength of concrete at the age of 28 days (psi),

f_{cmo} is taken as 1450 psi.

The effect of temperature on the creep coefficient may be taken into account by modifying the age at loading t_o , however, since it was not considered in the analysis, details about it was not provided in this thesis.

C.1.6. GL 2000

This model was developed by Gardner and Lockman in 2001. In 2004, Gardner included minor modifications to some coefficients and to the strength development with time, and the latest version of model was obtained.

The model provides a procedure for calculating the creep coefficient for normal-strength concretes, concretes with mean compressive strength less than 11890 psi. Apart from that, according to Gardner and Lockman (2001), it was indicated that this model can be used for any concrete specimens regardless of what chemical admixtures or mineral are in the concrete, casting temperature or curing regime.

$$\phi(t, t_o) = \phi_{28}(t, t_o) \frac{E_{cmto}}{E_{cm28}} \quad \text{C-50}$$

$$\phi_{28}(t, t_o) = \phi(t_c) \beta(t, t_o) \quad \text{C-51}$$

$$\beta(t, t_o) = \left[\frac{2(t - t_o)^{0.3}}{(t - t_o)^{0.3} + 14} + \left[\frac{7(t - t_o)}{t_o[(t - t_o) + 7]} \right]^{0.5} + 2.5(1 - 1.086h^2) \left[\frac{(t - t_o)}{(t - t_o) + 77(V/S)^2} \right]^{0.5} \right] \quad \text{C-52}$$

$$\text{If } t_o = t_c \quad \phi(t_c) = 1 \quad \text{C-53}$$

$$\text{If } t_o > t_c \quad \phi(t_c) = \left[1 - \left[\frac{(t - t_o)}{(t - t_o) + 77(V/S)^2} \right]^{0.5} \right]^{0.5} \quad \text{C-54}$$

Where:

$\phi(t, t_o)$ is the creep coefficient function,

$\phi_{28}(t, t_o)$ is the 28-day creep coefficient,

$\beta(t, t_o)$ is the coefficient describing the development of creep with time,

$\phi(t_c)$ is the correction term for the effect of drying before loading,

V/S is the volume-to-surface ratio (in.),

E_{cm28} is the mean modulus of elasticity (psi),

E_{cmo} is mean the modulus of elasticity at the time of loading, t_o , (psi),

t is the age of concrete (days),

t_c is the age drying of concrete starts or end of moist curing (days),

t_o is the age of concrete at loading (days).

C.1.7. fib MC 2010

The range of applicability of this model is as follows:

- Unless special provisions are provided, it is applicable for ordinary structural concrete whose mean strength is in between 2900 psi and 18850 psi.

- Concrete compressive stress level should not exceed 40 % of its mean compressive strength at the time of loading.
- The ambient relative humidity should be at least 40 %.
- The temperature range should be in between 41 °F and 86 °F.
- The age of loading should be at least 1 day.

$$\phi(t, t_o) = \phi_{bc}(t, t_o) + \phi_{dc}(t, t_o) \quad \text{C-55}$$

The basic creep coefficient, $\phi_{bc}(t, t_o)$, may be estimated from:

$$\phi_{bc}(t, t_o) = \beta_{bc}(f_{cm}) \cdot \beta_{bc}(t, t_o) \quad \text{C-56}$$

$$\beta_{bc}(f_{cm}) = \frac{1.8}{(f_{cm}/145)^{0.7}} \quad \text{C-57}$$

$$t_{o,adj} = t_o \left[\left(\frac{9}{2 + t_o^{1.2}} \right) + 1 \right]^\alpha \quad \text{C-58}$$

$$\beta_{bc}(t, t_o) = \ln \left[\left(\frac{30}{t_{o,adj}} + 0.035 \right)^2 (t - t_o) + 1 \right]$$

The drying creep coefficient, $\phi_{dc}(t, t_o)$, may be estimated from:

$$\phi_{dc}(t, t_o) = \beta_{dc}(f_{cm}) \cdot \beta(RH) \cdot \beta_{dc}(t_o) \cdot \beta_{dc}(t, t_o) \quad \text{C-59}$$

$$\beta_{dc}(f_{cm}) = \frac{412}{(f_{cm}/145)^{1.4}} \quad \text{C-60}$$

$$\beta(RH) = \frac{1 - \frac{RH}{100}}{\sqrt[3]{\frac{h}{25400}}} \quad \text{C-61}$$

$$h = \frac{2A_c}{u} \quad \text{C-62}$$

$$\beta_{dc}(t_o) = \frac{1}{0.1 + (t_{o,adj})^{0.2}} \quad \text{C-63}$$

$$\beta_{dc}(t, t_o) = \left[\frac{(t - t_o)}{\beta_h + (t - t_o)} \right]^{\gamma(t_o)} \quad \text{C-64}$$

$$\gamma(t_o) = \frac{1}{2.3 + \frac{3.5}{\sqrt{t_{o,adj}}}} \quad \text{C-65}$$

$$\beta_h = 38.1h + 250\alpha_{f_{cm}} \leq 1500\alpha_{f_{cm}} \quad \text{C-66}$$

$$\alpha_{f_{cm}} = \left(\frac{35}{f_{cm}/145} \right)^{0.5} \quad \text{C-67}$$

Where:

t is the age of concrete at the moment considered (days),

t_0 is the age of concrete at loading (days),

$t_{o,adj}$ is the age of concrete at loading adjusted for cement type only (days),

f_{cm} is the mean compressive strength at an age of 28 days (psi),

α is a power that depends on the type of cement (Table C.1),

RH is the relative humidity of the ambient environment (%),

A_c is the cross-sectional area (in.²),

u is the perimeter of the member in contact with the atmosphere (in.),

The effect of temperature on the creep coefficient may be taken into account by modifying the age at loading t_o , however, since it was not considered in the analysis, details about it was not provided in this thesis.

C.2. Shrinkage Strain Prediction Models

Similar to creep models, there are 7 main shrinkage models investigated in this thesis: AASHTO LRFD 2020, ACI 209R-92, Bažant-Baweja B3, CEB MC90, CEB MC90-99, GL 2000 and fib MC 2010. Out of these, AASHTO LRFD 2020 is divided into two separate shrinkage models: AASHTO Body 2020 and AASHTO Commentary 2020. Therefore, in total eight different shrinkage models are explored in this thesis.

C.2.1. AASHTO LRFD 2020

Similar to creep, this model should also be used for calculating prestressing loss in bridges other than segmentally constructed ones (AASHTO LRFD 2020). Furthermore, code allows it to use also for determining the effects of shrinkage in deflection calculations. There are four different factors which are needed to be considered while calculating the shrinkage strain.

$$\varepsilon_{sh} = k_s k_{hs} k_f k_{td} 0.48(10^{-3}) \quad \text{C-68}$$

$$k_{sb} = 1.45 - 0.13(V/S) \geq 1.0 \quad \text{C-69}$$

$$k_{sc} = \left[\frac{\frac{t}{26e^{0.36(V/S)} + t}}{\frac{t}{45 + t}} \right] \left[\frac{1064 - 94(V/S)}{923} \right] \quad \text{C-70}$$

$$k_{hs} = 2.00 - 0.014H \quad \text{C-71}$$

$$k_f = \frac{5}{1 + f'_{ci}} \quad \text{C-72}$$

$$k_{td} = \frac{t}{12 \left(\frac{100 - 4f'_{ci}}{f'_{ci} + 20} \right) + t} \quad \text{C-73}$$

Where:

ε_{sh} is the shrinkage strain,

H is the average annual ambient relative humidity (%),

k_s is factor for the effect of the volume-to-surface ratio of the component,

k_{hs} is humidity factor for shrinkage,

k_f is factor for the effect of concrete strength,

k_{td} is the time development factor,

t is the maturity of concrete, defined as age of concrete between end of curing and time at being considered for the analysis of shrinkage effects (days),

V/S is the volume-to-surface ratio (in.),

f'_{ci} is the design concrete compressive strength at the time of load application (ksi).

It should be noted that if the concrete is exposed to drying before 5 days of curing have elapsed, the shrinkage strain should be increased by 20 percent (AASHTO LRFD 2020).

As it can be seen above, there are two equations for the effect of volume-to-surface ratio: the first one is mentioned in the body of AASHTO LRFD 2020, and second one is mentioned in the commentary side of the AASHTO LRFD 2020. The shrinkage strains have been calculated by using these two approaches separately.

Moreover, similar to creep, the model is applicable for the design concrete compressive strengths up to 15 ksi. Furthermore, if the concrete age is not known at the time of loading, f'_{ci} may be taken as 80 % of 28 day concrete compressive strength, f'_c .

C.2.2. ACI 209R-92

Similar to creep formulae of this model, ultimate shrinkage strain, ϵ_{shu} , is estimated by considering 7 different parameters. Then, a time-rate function, the first part of the C-74 below, is utilized to predict the development of shrinkage strain with time.

$$\epsilon_{sh}(t, t_c) = \frac{(t - t_c)^\alpha}{f + (t - t_c)^\alpha} \epsilon_{shu} \quad \text{C-74}$$

Where:

$\epsilon_{sh}(t, t_c)$ is the shrinkage strain at concrete age t ,

f (days) and α are constants for a given member shape,

ϵ_{shu} is the ultimate shrinkage strain,

t is the age of concrete (days),

t_c is the number of curing days (days).

It is suggested that in the absence of specific shrinkage data for local aggregates and conditions and at ambient relative humidity of 40 %, the average value for ϵ_{shu} may be taken as $780 \mu\epsilon$. Moreover, the value f may be taken as 35 and 55 for 7 days of moist curing and 1 to 3 days of steam curing, respectively, and α may be taken as 1.0 for flatter curve. On the other hand, the effect of member shape and size could be fully considered by replacing f with C-8.

For other than standard conditions stated in Figure C.1, the ultimate shrinkage strain, ϵ_{shu} , needs to be modified by 7 factors.

$$\varepsilon_{shu} = \gamma_{sh} 780(10^{-6}) \quad C-75$$

$$\gamma_{sh} = \gamma_{sh,tc} \gamma_{sh,RH} \gamma_{sh,vs} \gamma_{sh,s} \gamma_{sh,\psi} \gamma_{sh,c} \gamma_{sh,\alpha} \quad C-76$$

Where:

γ_{sh} is the cumulative product of the applicable correction factors,

$\gamma_{sh,tc}$ is the initial moist curing coefficient,

$\gamma_{c,RH}$ is the relative humidity correction factor,

$\gamma_{c,vs}$ is the volume-to-surface correction factor,

$\gamma_{c,s}$ is the slump correction factor,

$\gamma_{c,\psi}$ is the fine aggregate correction factor,

$\gamma_{sh,c}$ is the cement content factor,

$\gamma_{sh,\alpha}$ is the air content correction factor.

For the initial moist curing coefficient, $\gamma_{sh,tc}$, a table and closed form regression based equation are provided. In this thesis, it is calculated by using the equation below. It should be noted that the correction factor $\gamma_{sh,tc}$ may be taken as 1.0 for steam curing with period of 1 to 3 days. Other than that, the model is not applicable.

$$\text{For moist curing:} \quad \gamma_{sh,tc} = 1.202 - 0.2337 \log(t_c) \quad C-77$$

$$\begin{aligned} &\text{For steam curing} \\ &\text{(1-3 days):} \quad \gamma_{sh,tc} = 1.0 \quad C-78 \end{aligned}$$

Where:

t_c is number of moist cured days (days).

The ambient relative humidity factor, $\gamma_{sh,RH}$, is

$$\text{For } 0.80 \geq h \geq 0.40: \quad \gamma_{sh,RH} = 1.40 - 1.02h \quad \text{C-79}$$

$$\text{For } 1 \geq h \geq 0.80: \quad \gamma_{sh,RH} = 3.00 - 3.0h \quad \text{C-80}$$

$$\text{For } 0.40 > h: \quad \gamma_{sh,RH} > 1.0 \quad \text{C-81}$$

Where:

h is relative humidity in decimals.

It is indicated that if h is lower than 0.40, value for $\gamma_{sh,RH}$ higher than 1.0 should be used.

The size correction factor of the member, $\gamma_{sh,vs}$ is

$$\gamma_{sh,vs} = 1.2e^{-0.12(V/S)} \quad \text{C-82}$$

Where:

V is the volume of the member (in.³),

S is the surface area of the member (in.²).

There is also alternative method presented in the ACI 209R-92, however, since it is was utilized in the analysis, it was not mentioned in this thesis.

The correction factor for slump of concrete, $\gamma_{sh,s}$, is

$$\gamma_{sh,s} = 0.89 + 0.041s \quad \text{C-83}$$

Where:

s is the slump of concrete (in.),

The correction factor fine aggregate, $\gamma_{sh,\psi}$ is

$$\text{For } \psi \leq 50 \%: \quad \gamma_{sh,\psi} = 0.30 + 0.014\psi \quad \text{C-84}$$

$$\text{For } \psi > 50 \%: \quad \gamma_{sh,\psi} = 0.90 + 0.002\psi \quad \text{C-85}$$

Where:

ψ is the ratio of fine aggregate to total aggregate by weight (%).

The correction factor for cement type, $\gamma_{sh,c}$, is

$$\gamma_{sh,c} = 0.75 + 0.00036c \geq 1 \quad \text{C-86}$$

Where:

c is the cement content (lb/yd^3).

The air content factor, $\gamma_{sh,\alpha}$, is

$$\gamma_{sh,\alpha} = 0.95 + 0.008\alpha \geq 1 \quad \text{C-87}$$

C.2.3. Bažant-Baweja B3

The restrictions for this model explained in section C.1.3 are also applicable in this part. The approach of this model somewhat similar to ACI 209R-92 approach such that in this model, a mean shrinkage strain is estimated, and multiplied by time curve and a factor.

$$\varepsilon_{sh}(t, t_c) = -\varepsilon_{sh\infty} k_h S(t - t_c) \quad \text{C-88}$$

$$\varepsilon_{sh\infty} = -\varepsilon_{s\infty} \frac{E_{cm607}}{E_{cm(t_c + \tau_{sh})}} \quad \text{C-89}$$

$$\varepsilon_{s\infty} = \alpha_1 \alpha_2 (0.02565 w^{2.1} f_{cm28}^{-0.28} + 270) (10^{-6}) \quad \text{C-90}$$

$$S(t - t_c) = \tanh \sqrt{\frac{(t - t_c)}{\tau_{sh}}} \quad \text{C-91}$$

$$\tau_{sh} = 190.8 t_c^{-0.08} f_{cm28}^{-0.25} [2k_s(V/S)]^2 \quad \text{C-92}$$

Table C.2. Factor for α_1 and α_2

Type of cement	α_1	Curing method	α_2
Type 1	1	Steam cured	0.75
Type 2	0.85	Cured in water or at 100 % relative humidity	1
Type 3	1.1	Sealed during curing or normal curing in air with initial protection against drying	1.2

Where:

$\varepsilon_{sh}(t, t_c)$ is the mean shrinkage strain in the cross-section at age of concrete t (days), measured from the start of drying at t_c (days),

$\varepsilon_{sh\infty}$ is the ultimate shrinkage strain,

$S(t - t_c)$ is the time curve,

$(t - t_c)$ is the time from the end of the initial curing (days).

$\varepsilon_{s\infty}$ is a constant,

$\frac{E_{cm607}}{E_{cm(t_c+\tau_{sh})}}$ is a factor that accounts for the time dependence of ultimate shrinkage, where

E_{cm607} and $E_{cm(t_c+\tau_{sh})}$ are mean modulus of elasticity at 607 days and at $t_c + \tau_{sh}$ days,

w is the water content (lb/yd^3),

f_{cm28} is the mean compressive strength at 28 days (psi),

α_1 and α_2 are the constants related to cement type and curing condition (Table C.2),

t and t_c are the age of concrete and the age drying commenced, respectively (days),

τ_{sh} is the shrinkage half-time (days),

k_s is the cross-section shape correction factor, may be taken as 1.0,

V/S is the volume-to-surface ratio (in.).

C.2.4. CEB MC90

The restrictions explained in section C.1.4 are also applicable for shrinkage calculations of this model. Like other approaches, this model predicts the shrinkage strain by providing an ultimate shrinkage strain with a time function.

$$\varepsilon_{sh}(t, t_c) = \varepsilon_{cso} \beta_s(t - t_c) \quad \text{C-93}$$

$$\varepsilon_{cso} = \varepsilon_s(f_{cm28}) \beta_{RH}(h) \quad \text{C-94}$$

$$\varepsilon_s(f_{cm28}) = \left[160 + 10\beta_{sc} \left(9 - \frac{f_{cm28}}{f_{cm0}} \right) \right] (10^{-6}) \quad \text{C-95}$$

$$\text{For } 0.4 \leq h < 0.99 \quad \beta_{RH}(h) = -1.55 \left[1 - \left(\frac{h}{h_o} \right)^3 \right] \quad \text{C-96}$$

$$\text{For } h \geq 0.99 \quad \beta_{RH}(h) = 0.25 \quad \text{C-97}$$

$$\beta_s(t - t_c) = \left[\frac{(t - t_c)/t_1}{350[(V/S)/(V/S)_o]^2 + (t - t_c)/t_1} \right]^{0.5} \quad \text{C-98}$$

Table C.3. Coefficient β_{sc}

Type of cement according to EC2	β_{sc}
SL (Slowly-hardening cements)	4
N and R (Normal or rapid hardening cements)	5
RS (rapid hardening high-strength cements)	8

Where:

$\varepsilon_{sh}(t, t_c)$ is the total shrinkage strain function,

ε_{cs0} is the notional shrinkage coefficient,

β_{sc} is a coefficient that depends on the cement type (Table C.3),

$\beta_{RH}(h)$ is a coefficient considering ambient relative humidity,

$\beta_s(t - t_c)$ is the time development function for shrinkage,

f_{cm28} is the mean compressive strength of concrete at the age of 28 days (psi),

f_{cm0} is taken as 1450 psi,

h is the relative humidity in decimals,

h_o is taken as 1.0,

V/S is the volume-to-surface ratio (in.),

$(V/S)_o$ is taken as 2 in.,

t_1 is taken as 1 day,

t is the age of concrete at the moment considered (days),

t_c is the age of concrete at the beginning of drying (days),

$(t - t_c)$ is the duration of drying (days).

The effect of temperature on the notional shrinkage coefficient may be taken into account by modifying the parameter $\beta_{RH}(h)$, however, since it was not considered in the analysis, details about it was not provided in this thesis.

C.2.5. CEB MC90-99

The restrictions explained in C.1.4 are also applicable for shrinkage calculations of this model. This model is the improved version of CEB MC90 model such that the shrinkage characteristics of high-performance concrete are considered. This model separates the total

shrinkage strain into two part: autogenous shrinkage and drying shrinkage. The drying shrinkage component is quite similar to one presented in CEB MC90 model with some adjustment factors.

$$\varepsilon_{sh}(t, t_c) = \varepsilon_{cas}(t) + \varepsilon_{cds}(t, t_c) \quad \text{C-99}$$

The autogenous shrinkage component $\varepsilon_{cas}(t)$, can be calculated as follows:

$$\varepsilon_{cas}(t) = \varepsilon_{caso}(f_{cm28})\beta_{as}(t) \quad \text{C-100}$$

$$\varepsilon_{caso}(f_{cm28}) = -\alpha_{as} \left(\frac{f_{cm28}/f_{cmo}}{6 + f_{cm28}/f_{cmo}} \right)^{2.5} (10^{-6}) \quad \text{C-101}$$

$$\beta_{as}(t) = 1 - e^{-0.2(t/t_1)^{0.5}} \quad \text{C-102}$$

The drying shrinkage component $\varepsilon_{cds}(t)$, can be calculated as follows:

$$\varepsilon_{cds}(t, t_c) = \varepsilon_{cdso}(f_{cm28})\beta_{RH}(h)\beta_{ds}(t - t_c) \quad \text{C-103}$$

$$\varepsilon_{cdso}(f_{cm28}) = \left[(220 + 110\alpha_{ds1})e^{-\alpha_{ds1} \frac{f_{cm28}}{f_{cmo}}} \right] (10^{-6}) \quad \text{C-104}$$

$$\text{For } 0.4 \leq h < 0.99 \beta_{s1} \quad \beta_{RH}(h) = -1.55 \left[1 - \left(\frac{h}{h_o} \right)^3 \right] \quad \text{C-105}$$

$$\text{For } h \geq 0.99 \beta_{s1} \quad \beta_{RH}(h) = 0.25 \quad \text{C-106}$$

$$\beta_{ds}(t - t_c) = \left[\frac{(t - t_c)/t_1}{350[(V/S)/(V/S)_o]^2 + (t - t_c)/t_1} \right]^{0.5} \quad \text{C-107}$$

$$\beta_{s1} = \left(\frac{3.5f_{cmo}}{f_{cm28}} \right)^{0.1} \leq 1.0 \quad \text{C-108}$$

Table C.4. Coefficients for CEB MC90-99 shrinkage model

Type of cement	α_{as}	α_{ds1}	α_{ds2}
SL (slowly- hardening cements) 32.5 N	800	3	0.13
N and R (normal or rapid hardening cements) 32.5 R, 42.5 N	700	4	0.12
RS (rapid hardening high-strength cement) 42.5 R 52.5 N, 52.5 R	600	6	0.12

Where:

$\varepsilon_{sh}(t, t_c)$ is the total shrinkage strain function,

$\varepsilon_{cds}(t, t_c)$ is the drying shrinkage function,

$\varepsilon_{cas}(t)$ is the autogenous shrinkage strain function,

$\varepsilon_{caso}(f_{cm28})$ is the notional autogenous shrinkage coefficient,

$\beta_{as}(t)$ is the function for time development of autogenous shrinkage,

$\varepsilon_{cdso}(f_{cm28})$ is the notional drying shrinkage coefficient,

$\beta_{ds}(t - t_c)$ is the function for time development of drying shrinkage,

α_{as} , α_{ds1} and α_{ds2} are coefficients depending on the type of cement (Table C.4),

β_{s1} is a coefficient that takes into account self-desiccation of high-performance concrete,

$\beta_{RH}(h)$ is a coefficient considering ambient relative humidity,

f_{cm28} is the mean compressive strength of concrete at the age of 28 days (psi),

f_{cmo} is taken as 1450 psi,

h is the relative humidity in decimals,

h_o is taken as 1.0,

V/S is the volume-to-surface ratio (in.),

$(V/S)_o$ is taken as 2 in.,

t_1 is taken as 1 day,

t is the age of concrete at the moment considered (days),

t_c is the age of concrete at the beginning of drying (days),

$(t - t_c)$ is the duration of drying (days).

C.2.6. GL 2000

As it is mentioned in the creep approach of this model, the only restriction for this model is 28-day mean compressive strength should not exceed 11890 psi. Similar to other shrinkage models, this model has composed of three terms: ultimate shrinkage strain, time development function and a correction factor for the effect of humidity.

$$\varepsilon_{sh}(t, t_c) = \varepsilon_{shu} \beta(h) \beta(t - t_c) \quad \text{C-109}$$

$$\varepsilon_{shu} = 900k \left(\frac{4350}{f_{cm28}} \right)^{0.5} (10^{-6}) \quad \text{C-110}$$

$$\beta(h) = 1 - 1.18h^4 \quad \text{C-111}$$

$$\beta(t - t_c) = \left[\frac{(t - t_c)}{(t - t_c) + 77(V/S)} \right]^{0.5} \quad \text{C-112}$$

Table C.5. Coefficient k

Cement Type	k
Type 1	1.0
Type 2	0.75
Type 3	1.15

Where:

$\varepsilon_{sh}(t, t_c)$ is the shrinkage strain function,

ε_{shu} is the ultimate shrinkage strain,

$\beta(h)$ is a correction term for the effect of ambient relative humidity,

$\beta(t - t_c)$ is the time development function,

f_{cm28} is the mean 28-day compressive strength (psi),

k is a shrinkage constant that depends on cement type (Table C.5)

h is the ambient relative humidity in decimals,

t is the age of concrete at time being considered (days),

t_c is the age of concrete when drying starts, or end of moist curing (days).

C.2.7. fib MC 2010

The range of applicability of this model is provided in the section C.1.7. This model has exactly same formulae with the CEB MC90-99 model. Therefore, formulae for this model will not be repeated here.

APPENDIX D: MDOTCamber Program User's Manual



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Disclaimer

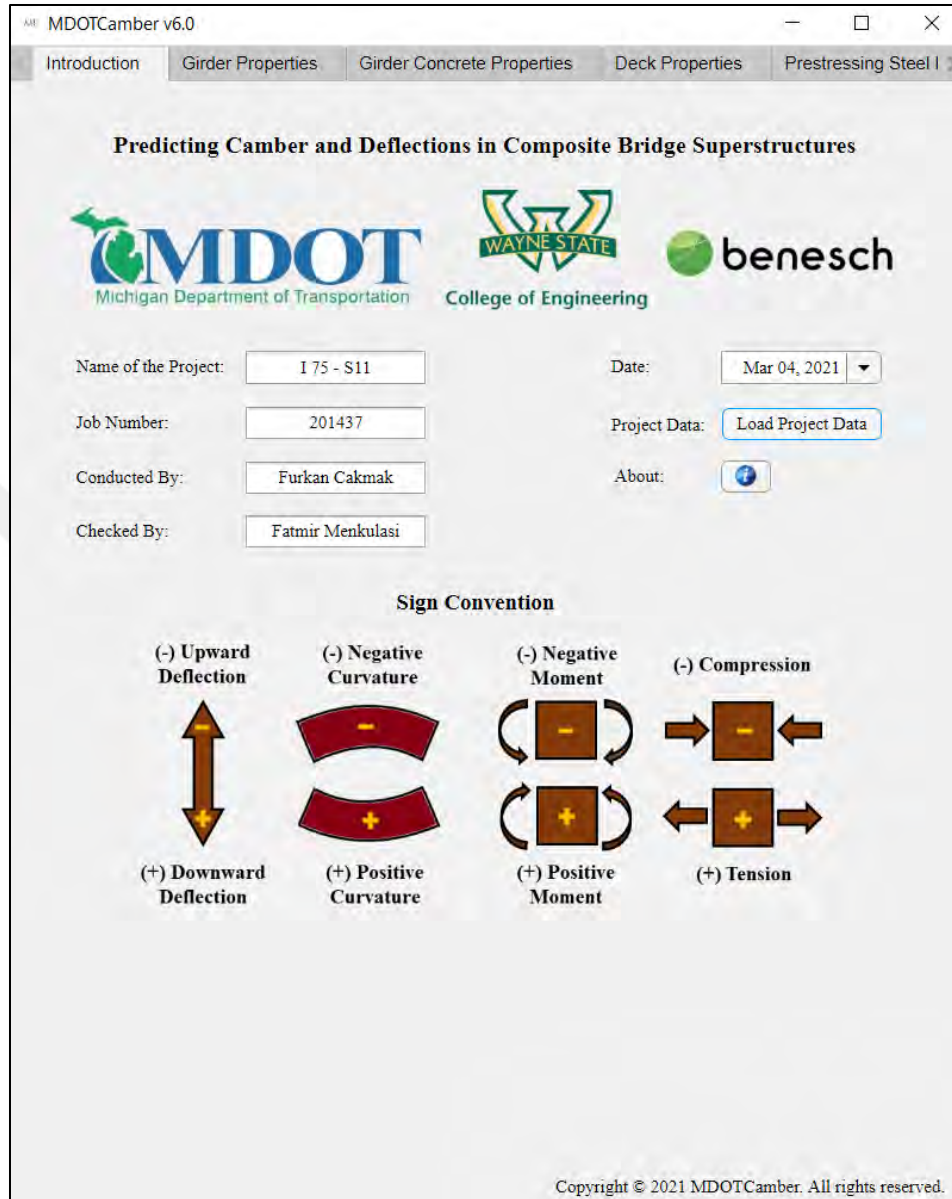
Though every effort has been devoted toward its accuracy, the software should not be used as a replacement for local standards of practice, design requirements, and engineering judgment. In no event shall the software developers or anyone distributing the software be liable for any damages or other liability arising from use of the software.

Software Limitations

The presented software is a tool to predict camber and displacements in fully prestressed concrete and steel beams when such beams are used in a simply supported configuration. The software considers non-composite fully prestressed concrete beams, and fully prestressed concrete and steel beams made composite with a cast in place concrete deck. The software considers one beam at a time. Continuous bridges superstructures are not addressed. For additional information refer to this manual as well as the final MDOT report titled "Evaluation of Camber and Deflections for Bridge Girders".

D.1. Tab 1: Introduction

In this page, it is required to provide general project information such as project name, job number, name of the person who conducts the analysis, name of the person who checks the analysis, and date of the analysis. In addition, previously saved input files can be loaded using the “Load Project Data” button. General information about how the program works is presented and the sign convention employed throughout the analysis is shown. In this program, upward deflection (camber), negative curvature and moment (tension in the top and compression in the bottom), and compression forces are shown as negative, whereas downward deflection, positive curvature and moment (compression in the top and tension in the bottom), and tension forces are shown as positive. It is important to consider this sign convention when providing a user defined input for the program such as pre-fabricated camber values for steel beams, or measured prestressed concrete beam camber values at specific time.



MDOTCamber v6.0

Introduction Girder Properties Girder Concrete Properties Deck Properties Prestressing Steel I >

Predicting Camber and Deflections in Composite Bridge Superstructures

MDOT Michigan Department of Transportation Wayne State College of Engineering benesch

Name of the Project: I 75 - S11 Date: Mar 04, 2021

Job Number: 201437 Project Data: Load Project Data

Conducted By: Furkan Cakmak About:

Checked By: Fatmir Menkulasi

Sign Convention

(-) Upward Deflection (+) Downward Deflection

(-) Negative Curvature (+) Positive Curvature

(-) Negative Moment (+) Positive Moment

(-) Compression (+) Tension

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Figure D.1. Introduction tab

D.2. Tab 2: Girder Properties

In this page, firstly, the beam type, either steel or prestressed concrete, and time-dependent analysis type should be selected as shown in Figure D.2. If the user selects to predict camber at service, then the boxes for camber at release and pre-erection camber are automatically selected. The user has the option to conduct the time-dependent analysis with or without including the effects of temperature gradients. When temperature analysis is intended to be performed, the

checkbox for this should be clicked and one information i.e. Zone I to IV, should be selected. These zones are described in AASHTO LRFD (2020) section 3.12.3. Since the inclusion of temperature gradients effects involves many numerical integrations, the analysis may take up to 120 seconds in Asus Laptop Computer with Intel Core i5 9300H processor with 2.3 GHz and 8 GB of RAM. If the inclusion of temperature gradient effects is omitted the analysis should be completed in less than 15 seconds in a similar computer. After selecting the beam type and analysis type, the button shown inside the red box in Figure D.2 must be clicked so that the selections are locked. Without locking the selections, the user cannot proceed with the analysis. If steel girder is selected as a beam type, the necessary input must be provided to before proceeding to the next step. Figure D.3 shows what these necessary inputs would be if a steel beam is selected.

The screenshot shows a software interface with two main sections. The left section, titled 'Select the Beam Type', contains two radio buttons: 'Steel Beam' and 'Prestressed Concrete Beam'. The right section, titled 'Analysis Type', contains three checkboxes: 'Temperature Gradient Analysis' (checked), 'Camber at Release' (checked), and 'Service Camber' (checked). There is also a 'Pre-erection Camber' checkbox which is unchecked. A 'Zone III' dropdown menu is visible next to the 'Temperature Gradient Analysis' checkbox. A red box highlights the 'Lock the selections made above' button at the bottom left.

Figure D.2. Selection of Beam Type and Analysis Type

If a prestressed concrete beam type is selected, a drop-down menu appears as shown in Figure D.4. This menu includes several standard girder cross-sections used by MDOT such as ASSHTO Type I-VI, MI 1800, Bulb Tees, and Box Beams. The menu includes three types of girder cross-section: I-shaped, Rectangular, and Box. If the girder to be used in the analysis matches with one of the standardized cross-sections, cross-sectional properties are automatically calculated. To see the automatically calculated values the button with the exclamation mark shown in Figure D.5 must be clicked. If the girder in question has a I-shape, rectangular shape, or box shape but does not match with one the standardized cross-sections, then the user has the option to enter custom cross-sectional properties.

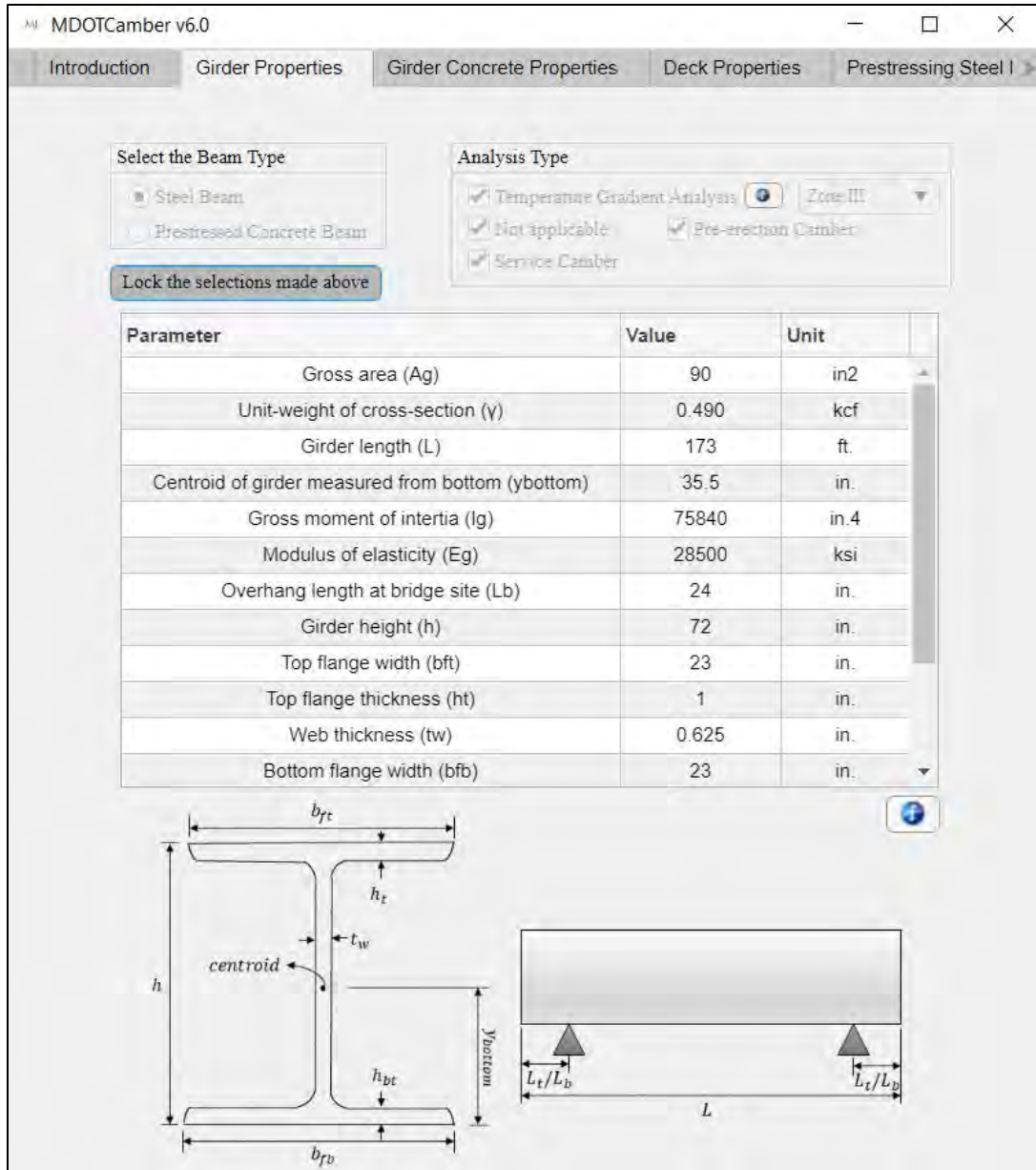


Figure D.3. Steel girder properties

MDOTCamber v6.0

Introduction Girder Properties Girder Concrete Properties Deck Properties Prestressing Steel I

Select the Beam Type

☐ Steel Beam

☒ Prestressed Concrete Beam

Lock the selections made above

Analysis Type

☒ Temperature Gradient Analysis ☒ Zone III

☒ Camber at Release ☒ Pre-erection Camber

☒ Service Camber

Select Girder Type

Select Girder Type

AASHTO Type I

AASHTO Type II

AASHTO Type III

AASHTO Type IV

AASHTO Type V

AASHTO Type VI

MI 1800

Bulb Tee 49" Top Flange Width - 36" Height

Bulb Tee 49" Top Flange Width - 42" Height

Bulb Tee 49" Top Flange Width - 48" Height

Bulb Tee 49" Top Flange Width - 54" Height

Bulb Tee 49" Top Flange Width - 60" Height

Bulb Tee 49" Top Flange Width - 66" Height

Bulb Tee 49" Top Flange Width - 72" Height

Bulb Tee 61" Top Flange Width - 36" Height

Bulb Tee 61" Top Flange Width - 42" Height

Bulb Tee 61" Top Flange Width - 48" Height

Bulb Tee 61" Top Flange Width - 54" Height

Bulb Tee 61" Top Flange Width - 60" Height

Bulb Tee 61" Top Flange Width - 66" Height

Figure D.4. Selection of girder type

MDOTCamber v6.0

Introduction Girder Properties Girder Concrete Properties Deck Properties Prestressing Steel I

Select the Beam Type

☐ Steel Beam

☒ Prestressed Concrete Beam

Lock the selections made above

Analysis Type

☒ Temperature Gradient Analysis ☒ Zone III

☒ Camber at Release ☒ Pre-erection Camber

☒ Service Camber

Bulb Tee 49" Top Flange Width - 54" Height

Parameter	Value	Unit
Unit-weight of cross-section* (γ)	0.15	kcf
Girder length (L)	136.9	ft.
Overhang length at precast facility (L_t)	21.5	in.
Overhang length at bridge site (L_b)	5.5	in.
Girder tributary width** (s)	8.41	ft.
Thermal expansion coefficient (α)	6.7e-06	1/F

Figure D.5. Girder properties for prestressed concrete girder

MDOTCamber v6.0

Parameter for Bulb Tee 49" Top Flange Width - 5...	Value	Unit
Gross area (A_g)	1022.3	in. ²
Centroid of girder measured from bottom (y_{bottom})	27	in.
Gross moment of inertia (I_g)	412056	in. ⁴
Girder height (h)	54	in.
Perimeter of cross-section (S_g)	251.4	in.
Top flange width (b_f)	49	in.
Top flange thickness (h_f)	5	in.
Web thickness (t_w)	8	in.
Height of top inclined surface (h_t)	3.51	in.

Figure D.6. Custom girder properties for bulb tee 49" top flange width – 54" height

D.3. Tab 3: Girder Concrete Properties

This page applies only to prestressed concrete girders and consists of three parts. The first involves the selection of a model to determine the variation of modulus of elasticity with time (Figure D.7). The user has two options: 1) ACI 209R-92, and 2) fib MC 2010. ACI 209R-92 is recommended solely because the camber data provided in the final report was evaluated based on the selection of this model as well as other applicable American models for predicting modulus of elasticity at prestress release and at 28 days, as well as creep and shrinkage models. While the use of these combination of models resulted in reasonable estimations of pre-erection camber when compared to measured data, the user is encouraged to evaluate other models with the purpose of obtaining a range of camber and displacements estimations and be prepared to accommodate this range of variation.

The second part requires the specification of compressive strength at prestress release and 28 days (Figure D.8a and Figure D.8b). Here the user has the option of providing measured values, specified values and no overstrength factors, and adjusting the specified values with overstrength factors. Various options are provided for specifying overstrength factors including Michigan overstrength factors that were derived based on compressive strength data analyzed as part of this research. The specification of compressive strength at release and 28 days is required to determine the modulus of elasticity at release and 28 days, which is the focus of the third part (Figure D.9). The modulus at 28 days serves as an anchor point to obtain the development of modulus with time, which is obtained based on the model selected in the first part (Figure D.9b). The user has the option of either entering measured values of modulus at release and 28 days, which is uncommon, but possible in special cases, or the option of selecting a model that relates compressive strength at release and 28 days to modulus at these two times (Figure D.9a). The models available for

relating compressive strength to modulus are AASHTO LRFD 2020, fib MC 2010, ACI 318-19, and ACI 363R-92. It is recommended that the AASHTO LRFD 2020 model is selected and since this model together with other applicable models was evaluated as part of this research effort and resulted in reasonable predictions of pre-erection camber and displacements when compared to measured values. Although, as indicated above the user is encouraged to explore the use of other models to get an idea about the range of variation in results. The evaluations of other models and their combinations was outside the scope of this research. Even if such a comparison is made, no definitive conclusion can be drawn as the results are a function of the dataset considered. Once a model is selected the user has the option to either select certain parameters to determine the variation of modulus with time based on the model selected in part 1, or, to back calculate these parameters using the modulus at release and 28 days as anchor points (Figure D.9b). The latter option is recommended.

A. Variation of Modulus of Elasticity with Time

ACI 209R-92 (Recommended) ▼ ⓘ

B. Concrete Compressive

☐ Click to provide actual concrete

Select a Model

ACI 209R-92 (Recommended)

fib MC 2010

8 Days

Figure D.7. Model selection for development of concrete strength and modulus of elasticity

MDOTCamber v6.0

Introduction Girder Properties Girder Concrete Properties Deck Properties Prestressing Steel I

A. Variation of Modulus of Elasticity with Time

ACI 209R-92 (Recommended) ⓘ

B. Concrete Compressive Strength at Release and at 28 Days

☒ Click to provide actual concrete compressive strength values.

Actual Concrete Compressive Strength

Symbol	Definition	Value (ksi)
f_{ci}	Actual concrete compressive strength at release	7
f_c	Actual concrete compressive strength at 28 day	10

a)

B. Concrete Compressive Strength at Release and at 28 Days

☐ Click to provide actual concrete compressive strength values.

Specified Concrete Compressive Strength

Symbol	Definition	Value (ksi)
f_{ci}	Specified concrete compressive strength at release	7
f_c	Specified concrete compressive strength at 28 day	10

Overstrength Factors for Concrete Strength

Michigan Factors (Recommended) ⓘ

Symbol	Definition	Factor
OS_i	Overstrength factor for concrete at release	1.2
OS_{28}	Overstrength factor for concrete at 28 day	1.2

b)

Figure D.8. a) Inputs for actual (measured) concrete compressive strength values at release and 28 days, and b) Inputs for specified concrete compressive strength and overstrength factors

C. Modulus of Elasticity at Release and at 28 Days

☒ Click to provide actual modulus of elasticity values.

Actual Modulus of Elasticity

Symbol	Definition	Value (ksi)
Eci	Actual modulus of elasticity at release	5000
Ec	Actual modulus of elasticity at 28 day	6000

Required Parameters for modulus of elasticity

☒ Back-calculate parameters. 

a)

C. Modulus of Elasticity at Release and at 28 Days

☐ Click to provide actual modulus of elasticity values.

Model for estimating elastic modulus: AASHTO LRFD 2020 (Recommended) 

Required Parameters for modulus of elasticity

☒ Back-calculate parameters. 

Unit-weight of plain concrete (wc): 0.15 kcf 

Aggregate factor (K1): 1

b)

Figure D.9. a) Inputs for measured Modulus of Elasticity values at release and 28 days, and b) Model for modulus of elasticity at release and 28 days

D.4. Tab 4: Deck Properties

If the user wishes to predict camber and displacements at service, then this page can be used to enter parameters related to deck properties. If the scope of the analysis is limited to pre-erection camber, then no deck parameters need to be specified. The program has the capability to consider the effect of existing deck removal and new deck placement on prestressed concrete beam

camber and displacements. If project in question does not involve deck replacement, only the properties for the existing deck need to be entered. If the project includes deck replacement, then the user must specify the properties of the new deck to predict net camber and displacements after the placement of the new deck. Additionally, it should be noted that unlike in the girder concrete properties tab, only the concrete compressive strength at 28 days is required for the deck. It is assumed that deck concrete compressive strength at the end of curing period is assumed to be 80 % of its 28 day value, and the AASHTO LRFD 2020 model is selected to estimate the deck concrete modulus of elasticity at 28 days. Also, the development of deck concrete modulus with time is based on the ACI 209R-92 model assuming Type I cement and moist curing. Since deck concrete compressive strength and modulus are not as influential as girder concrete compressive strength and modulus on camber and displacements, it was decided to use only the models specified above. In addition, the program has a capability to provide the slab and screed elevations if necessary inputs are provided as shown in Figure D.11. If slab and screed elevations are not intended to be obtained, then the parameters shown in Figure D.11 may be assigned to 0.

Has the deck been replaced? 

☒ Yes
☐ No

Parameter for the First Deck	Value	Unit
Specified concrete compressive strength at 28 day (f_{cd})	4	ksi
Unit-weight of deck and haunch* (with mild steel) (γ)	0.15	kcf
Average deck thickness** (td)	9	in.
Aggregate factor* (K_1)	1	-
Unit weight of plain concrete* (w_c)	0.145	kcf
Average haunch thickness (th)	0	in.
Overlay thickness (to)	2	in.
Unit-weight of overlay (γ_o)	0.14	kcf
Barrier load^^ (wb)	0.04	klf

Parameter for the Second Deck	Value	Unit
Specified concrete compressive strength at 28 day (f_{cd})	4	ksi
Unit-weight of deck and haunch* (with mild steel) (γ)	0.15	kcf
Average deck thickness** (td)	9	in.
Aggregate factor* (K_1)	1	-
Unit weight of plain concrete* (w_c)	0.145	kcf
Average haunch thickness (th)	0	in.
Overlay thickness (to)	2	in.
Unit-weight of overlay (γ_o)	0.14	kcf
Barrier load^^ (wb)	0.04	klf

Figure D.10. Inputs for the decks



Parameter	Value	Unit
Slab formwork load (wsf)	10	psf
Surface load due to rebars (wsr)	4	psf
Diaphragm load (Pd)	5	kips
Number of interior diaphragms* (Ndp)	1	-

Figure D.11. Inputs for computation of slab and screed elevations

D.5. Tab 5: Prestressing Steel Properties

In this page, the user specifies the properties and configuration of prestressing strands. This information is provided in three tables. The first table includes information about cross-sectional and mechanical properties. The second and third table include information regarding the configuration of strands. For example, in the second layer the user has the option to specify bonded as well as debonded strands including the length of debonding. The third table provides the opportunity to specify the configuration of any harped strands including harping point and eccentricities at the end and midspan. The following additional explanation can be found by clicking the button with the exclamation mark.

- a. Layer: It is a set of strands positioned at the same elevation.
- b. Layer Group: It is a subset of strands within a Layer with identical harping or debonding configurations.
 - Fully Bonded, Straight Strands (Table 2)
 - Debonded, Straight Strands (Table 2)
 - Harped Strands (Table 3)
- c. Layer Group Detail Length = debonded length
- d. Hold-down location from girder end = harping point
- e. Distance from bottom of girder represents the distance from the bottom of girder to centroid of a layer group.
- f. To add or remove a Layer Group, the +/- buttons next to the tables shall be used.
- g. Nominal diameter, d_{ps} , for strands must be provided as a decimal value, and shall be selected from the available options: 0.375 in., 0.4375 in., 0.5 in., 0.5625 in., 0.60 in., 0.62 in., and 0.70 in..

- h. For the strand type, the input value must be either 1 or 2 for low-relaxation and stress-relieved strand types, respectively.
- i. To perform the analysis, at least one layer group must be fully bonded. It can be either fully bonded straight or harped. In other words, analysis with only debonded strands cannot be conducted.

☒ Check if there is any harped strand. 

Parameter	Value	Unit
Modulus of elasticity (Eps)	28500	ksi
Nominal diameter (dps)	0.6	in.
Ultimate tensile strength (fpu)	270	ksi
Jacking stress (fpj)	202.5	ksi
Yield stress (fpy)	243	ksi
Strand Type	1	1:Low-r...

Layer Group	Group Type	Number of Strands in Layer Group	Distance from Bottom of Girder (in.)	Debonded Length (ft.)
1	Fully Bonded, Straight	14	2	0
2	Fully Bonded, Straight	16	4	0
3	Fully Bonded, Straight	12	6	0
4	Fully Bonded, Straight	8	8	0
5	Fully Bonded, Straight	2	10	0
6	Debonded, Straight	1	2	4
7	Debonded, Straight	2	2	12

Layer Group	Group Type	Number of Strands in Layer Group	Distance from Bottom of Girder at Midspan (in.)	Distance from Bottom of Girder at End (in.)	Harping Point (ft.)
1	Harped	12	7	46	44.77

Figure D.12. Inputs for prestressing steel properties

D.6. Tab 6: Creep and Shrinkage Model Selection

In this page, the user can select one of out of eight creep and shrinkage models (Figure D.13), which would be used for beam and deck concrete. The AASHTO Body 2020 model is recommended as this is the model that was evaluated as part of this research and together with the other selected models resulted in reasonable estimations of pre-erection camber when compared to measured values. As noted earlier, the user is encouraged to evaluate all models to get a sense for the anticipated variability in camber and displacement predictions. The model labeled as AASHTO Body 2020 is based on creep the shrinkage models provided in AASHTO LRFD Bridge Design Specifications (2020) using equations presented in the body of AASHTO. AASHTO Commentary means that the creep and shrinkage models are based on alternative parameters provided in the commentary of AASHTO. Depending on the selected model type, additional parameters about properties of concrete mixture such as water content, air content, and cement content may be required. The default parameters were taken from MDOT project (I75-S11 Job No. 201437). The user should specify the applicable values for the project in question (if they are known). Additional information can be obtained by clicking the “i” (information) buttons or by exploring section 3.7 of this thesis.

Note: After the model is selected, the parameters corresponding to the selected model should be provided. If they are not known, default values (shown below if appropriate) will be assigned to them.

Creep and Shrinkage Model: AASHTO Body 2020 (Recommended) ▼ ⓘ

Parameter

Tensile creep magni ⓘ

Flexural creep magni ⓘ

Parameters needed to be defined:

AASHTO Body 2020

For this model, no parameter is required.

Select a Model

AASHTO Body 2020 (Recommended)

AASHTO Commentary 2020

ACI 209R-92

Bazant-Baweja B3

CEB MC90

CEB MC90-99

GL 2000

fib MC 2010

Figure D.13. Available creep and shrinkage models

D.7. Tab 7: Time, Environment and Measured Camber Properties

In this page, the user has the option to specify the timeline of certain construction activities and information regarding relative humidity and curing time (Figure D.14). The user has also the option to select the method for generating the time matrix. Two options are available: Proposed, and Gilbert and Ranzi (2010). For additional information regarding these two methods please explore section 8.6.6 or click the “i” button for a brief explanation. In addition, the user has the option of entering any measured camber data that may be available at a certain time (Figure D.15). If such data is entered, then the algorithm conducts camber and displacements predictions for any point in time after the time when such measurements were taken. It should be noted that the measured camber values should be entered based on the sign convention stated in Figure D.1. Additionally, the user should specify the number of beam sections to be considered in analysis. This number affects accuracy as well as the deflected shape of the beam. To obtain a smooth curve for the deflected shape at least 51 sections are recommended. Since analysis time highly depends on this input, values higher than 200 are not recommended as they are not necessary.

MDOTCamber v6.0

Creep and Shrinkage Model Selection | Time, Environment and Measured Camber Properties | Results

Number of sections to be investigated: 51

A. Time and Environment Properties

Parameter	Value	Unit
Relative humidity* (H)	70	%
Age of girder when prestressing strands are released (t1)	1	days
Age of girder at deck placement (t2)	28	days
Age of girder when barrier load is applied on first super-structure (t3)	35	days
Age of girder when overlay load is applied on first super-structure (t4)	35	days
Age of girder at deck removal (t5)	18250	days
Age of girder at new deck placement (t6)	18260	days
Age of girder when barrier load is applied on second super-structure (t7)	18267	days
Age of girder when overlay load is applied on second super-structure (t8)	18267	days

Time Matrix Generation

Proposed (Recommended) | Gilbert et al. (2010)

A logarithmic timescale will be assigned for conducting time-dependent calculations. Please consult user's manual for detailed information.

B. Measured Camber Properties

☒ Check if there is any measured camber value.

Provide measured camber input for a particular time

Figure D.14. Input to specify the timeline of certain construction activities and information regarding relative humidity and curing time; method to generate time matrix; and measured camber properties

Provide measured camber input for a particular time
Measured camber at release (Δ_{im})
Measured camber at pre-erection state (Δ_{pm})
Measured camber just after deck placement (Δ_{adp})
Measured camber just before application of barrier load (Δ_{babl})
Measured camber just after application of barrier load (Δ_{aab1})
Measured camber just before application of overlay load (Δ_{baol})
Measured camber just after application of overlay load (Δ_{aaol})
Measured camber just before deck removal (Δ_{bdr})
Measured camber just after deck removal (Δ_{adr})
Measured camber just before new deck placement (Δ_{bndp})
Measured camber just after new deck placement (Δ_{andp})
Measured camber just before application of barrier load (Δ_{babl2})
Measured camber just after application of barrier load (Δ_{aab12})
Measured camber just before application of overlay load (Δ_{baol2})
Measured camber just after application of overlay load (Δ_{aaol2})
Provide measured camber input for a particular time ▼

Figure D.15. Drop-down list for measured camber input

D.8. Tab 8: Results

In this page, the user can click on the “calculate” button to conduct the time-dependent analysis. Once the analysis is completed, the results will be presented as shown in Figure D.16. The first table shows the net camber not including and including the effects of positive and negative temperature gradients at several discrete times. This data can be downloaded as an excel file using the “Download Data” button. The figure shows the full camber and displacement history from the inception of prestressed concrete girders to the end of the service life of the bridge. The second table provides the data for the full camber and displacement history in a tabular format. This data

can also be downloaded as an excel file using the “Download Data” button. The user has also the option of calculating camber at a specific time by entering the desired time in days and by clicking the “Get Camber” button. The user can save the project data by clicking on the “Save Project Data” button. This greatly facilitates the re-running of the analysis if one of the parameters is decided to be changed at a later time. The results shown in this tab can be saved as an image file and later printed by clicking on the “Print Results” file.

In addition to this information, the user can obtain additional results by clicking on the “Detailed Analysis” button next to the “Calculate” button. This prompts the generation of a new window with two tabs: “Time Based Analysis” and “Location Based Analysis”. Clicking on the “Time Based Analysis” tab the user will prompt the generation of three figures (Figure D.17a). The first shows the full 3D camber and displacement history along the span of the beam starting from support to midspan. The second shows the variation of curvature with time at midspan, and the third shows the variation of prestress with time at midspan. Clicking on the “Location Based Analysis” tab will prompt the user to enter the time at which analysis is to be conducted (for example 1000 days). Providing this information and clicking on the “Calculate” button will prompt the generation of three figures and one table (Figure D.17b). The first shows the variation of curvature along the span at the specified time, the second shows the variation of camber along the span at the specified time (i.e. deflected shape), and the third shows the variation of prestress along the span at the specified time. The table provides the data that is used to plot the deflected shape for the beam for half of the span and can be downloaded by clicking the “Download Data” button.

Moreover, stress analysis for girder and deck at any time and location can be obtained by clicking “Stress Analysis” button. This opens a new window with three tabs: “Stress Analysis for Girder”, “Stress Analysis for Deck” and “Location Based Analysis”. Under the “Stress Analysis

for Girder” tab, user can see the two 3D graphs which shows the stress history for girder at top and bottom fibers, and one 2D graph which shows the variation in stresses at the top and bottom fibers for girder (Figure D.18a). The same results can also be obtained for the deck, or decks if there is deck replacement activity by selecting the “Stress Analysis for Deck” tab (Figure D.18b). Under “Location Based Analysis” tab, user can select a time to obtain the girder and deck stresses at the top and bottom fibers along the span (Figure D.19a and Figure D.19b). Moreover, cross-sectional stress analysis for given time can be obtained (Figure D.19c).

Finally, based on provided slab and screed elevation data under “Deck Properties” tab (Figure D.11), user can get the results for the determining slab and screed elevations by clicking “Slab and Screed Guidance” button. This button leads to open a window showing table or tables if there is deck replacement activity where user can get the camber and displacement data at specific events (Figure D.20). These data can be used to determine slab and screed elevations and can be downloaded by clicking “Download Data” buttons.

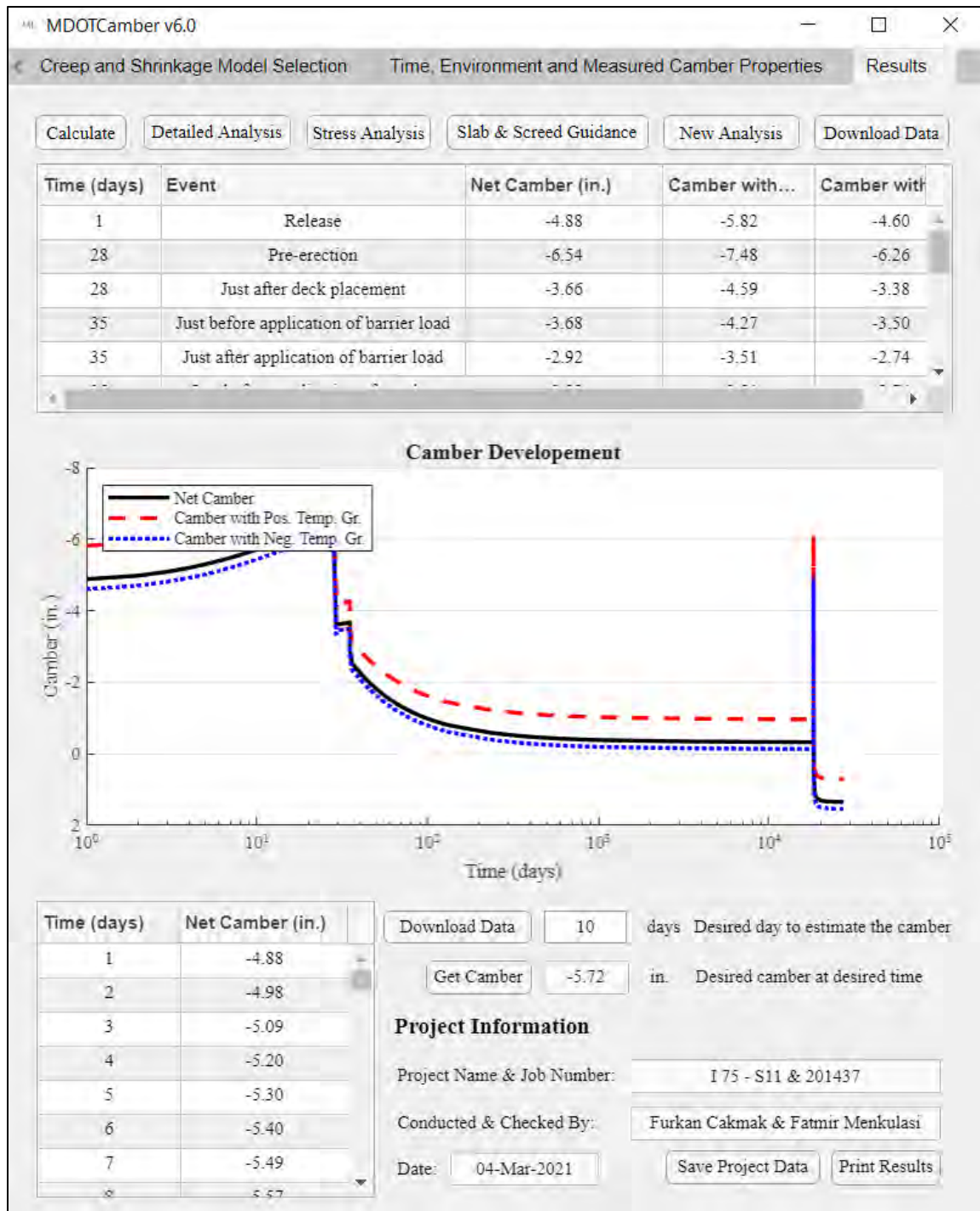
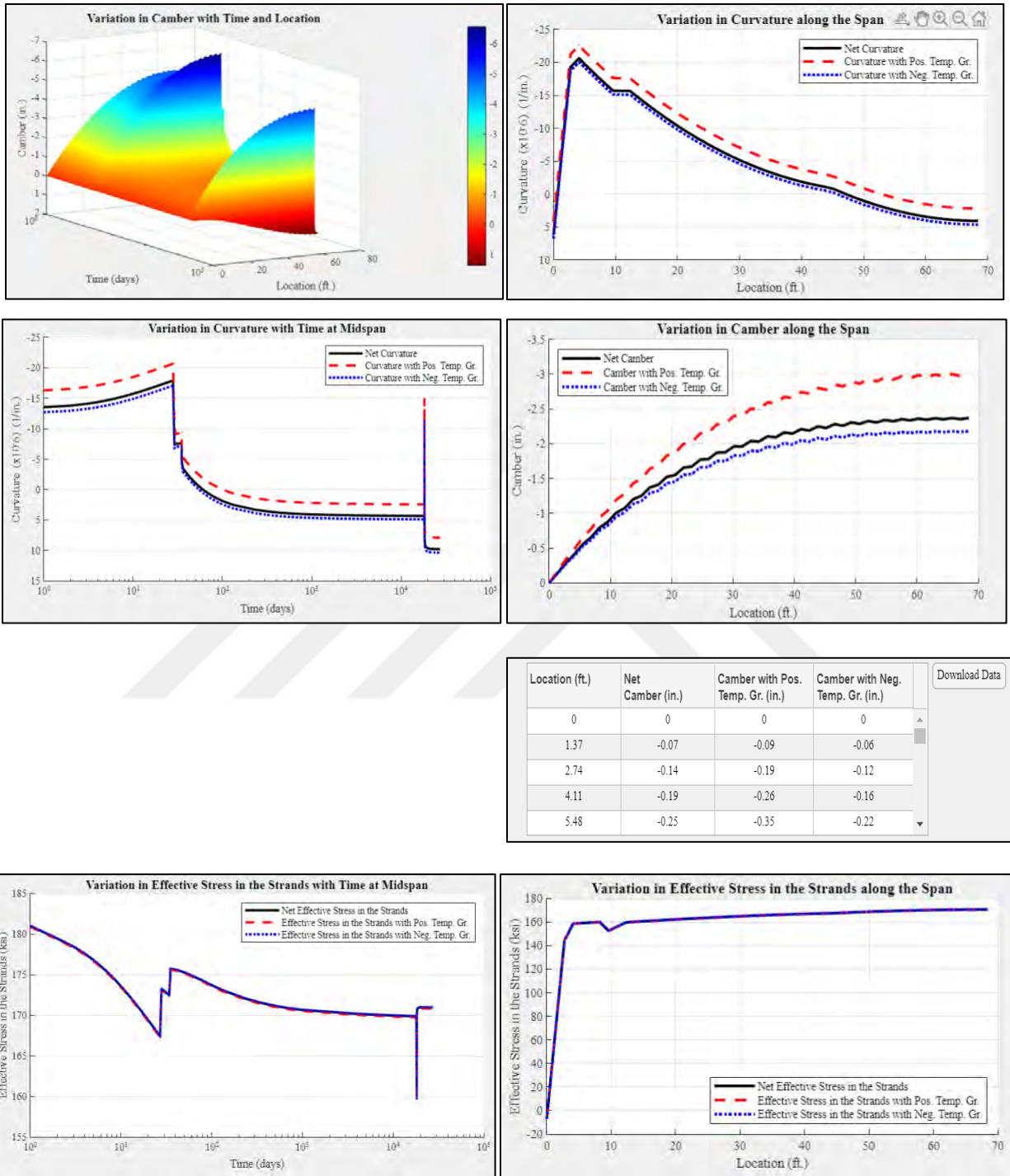


Figure D.16. Results of time-dependent analysis



a)

b)

Figure D.17. Detailed Analysis results: a) Time Based Analysis, b) Location Based Analysis

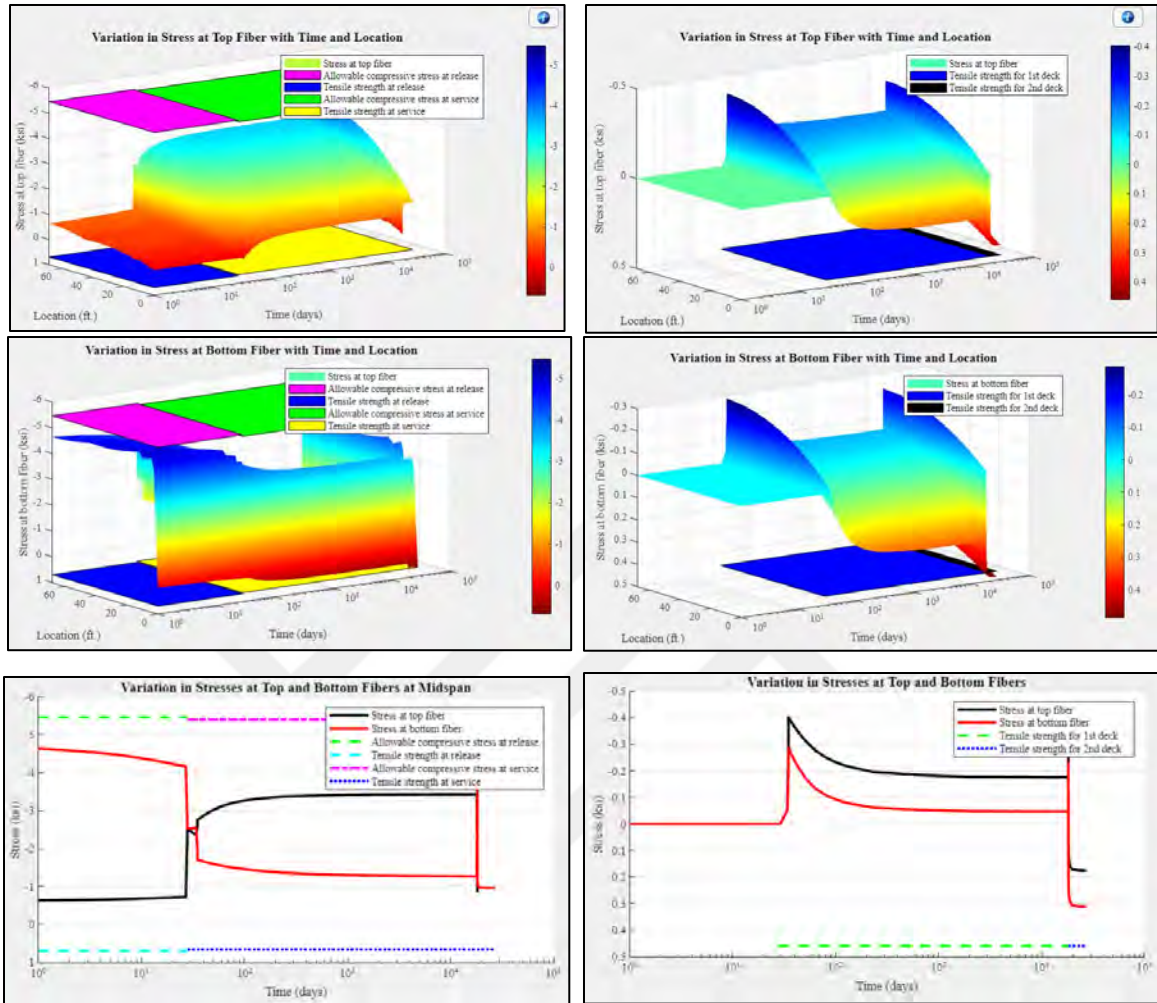


Figure D.18. Stress Analysis results: a) For girder, b) For decks

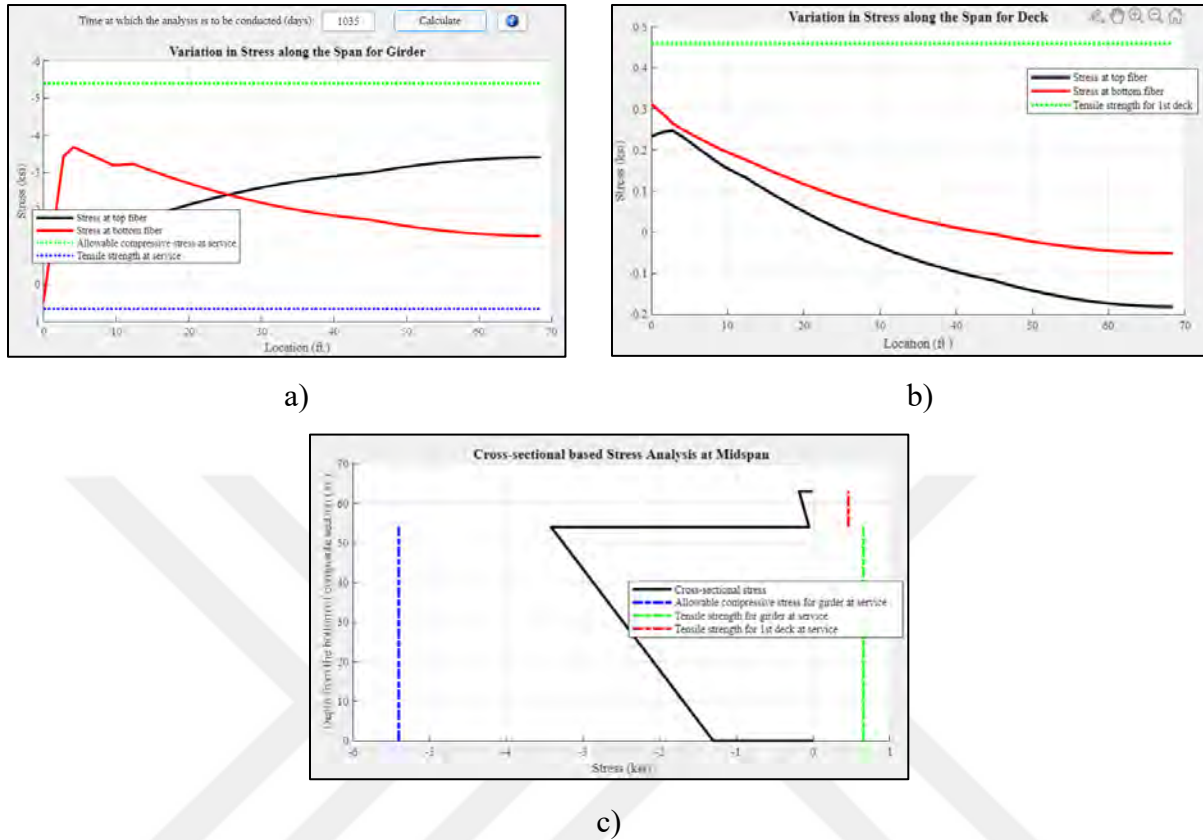
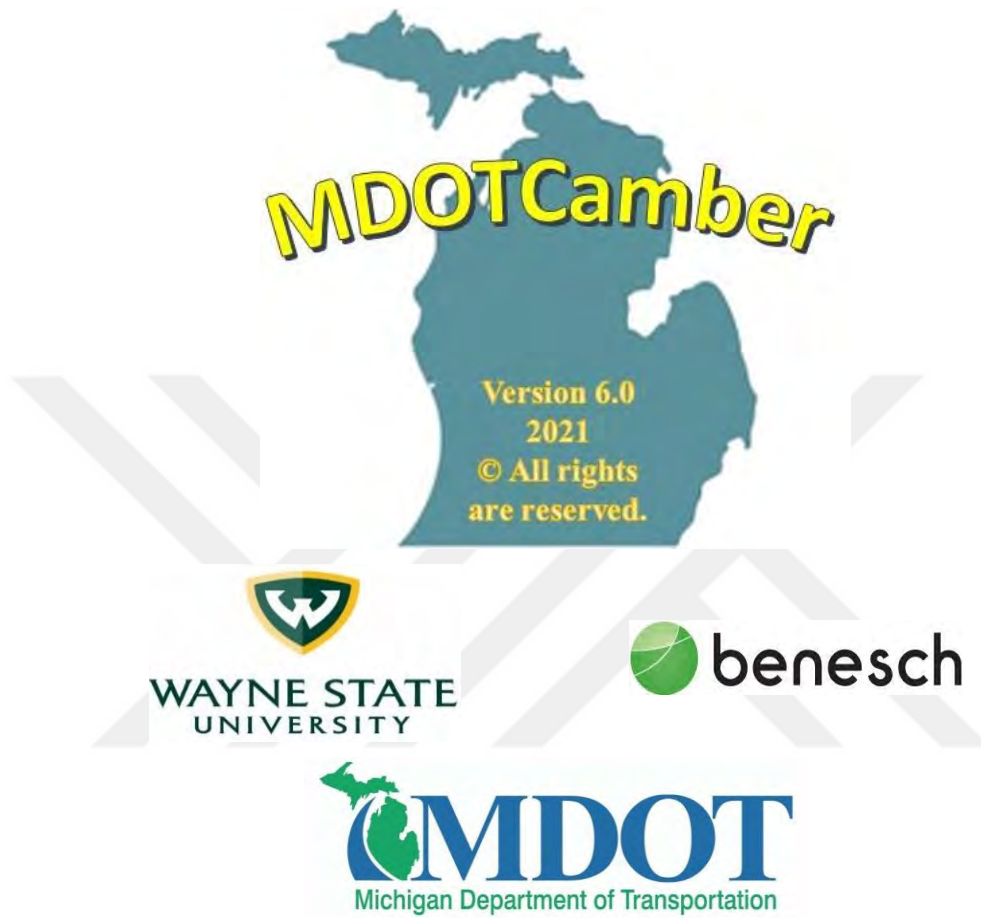


Figure D.19. Location based stress Analysis results at given time: a) For girder, b) For deck, and c) Along the cross-section

For the first super-structure					Download Data
Location (ft.)	Net Camber just after girder erection (in.)	Net Camber just after app. of formwork and deck rein. (in.)	Net Camber just after app. of diaphragm load (in.)	Net Camber just after app. of deck load (in.)	
0	0	0	0	0	▲
1.37	-0.30	-0.30	-0.30	-0.21	
2.74	-0.60	-0.59	-0.59	-0.42	
4.11	-0.88	-0.87	-0.87	-0.62	
5.48	-1.18	-1.17	-1.16	-0.83	
6.84	-1.43	-1.42	-1.42	-1.00	
8.21	-1.73	-1.71	-1.71	-1.30	▼
For the second super-structure					Download Data
Location (ft.)	Net Camber of girder alone on bridge site (in.)	Net Camber just after app. of formwork and deck rein. (in.)	Net Camber just after app. of diaphragm load (in.)	Net Camber just after app. of deck load (in.)	
0	0	0	0	0	▲
1.37	-0.30	-0.30	-0.29	-0.21	
2.74	-0.60	-0.59	-0.59	-0.42	
4.11	-0.87	-0.86	-0.86	-0.61	
5.48	-1.17	-1.16	-1.15	-0.82	
6.84	-1.42	-1.40	-1.40	-0.98	
8.21	-1.71	-1.60	-1.60	-1.10	▼

Figure D.20. Slab Screed Information

APPENDIX E: Example Problem for Proposed Prediction Methodology for Prestressed Concrete Girders



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E.1. Definition of the Problem

Using the proposed prediction methodology, Approach No. 6, explained in this thesis, camber and displacement along the span is going to be estimated at:

1. detensioning of the strands (t_1);
2. time at which the deck is placed (t_2);
3. time at placement of barriers and the opening of bridge to traffic (t_3);
4. time at placement of overlay (t_4);
5. time at deck removal for projects that features a deck replacement (t_5);
6. time at new deck placement (t_6);
7. time at placement of barrier and reopening of bridge to traffic (t_7);
8. time at placement of overlay (t_8);
9. time at the end of service life (t_9);

for the following prestressed concrete beam used in S11 project:

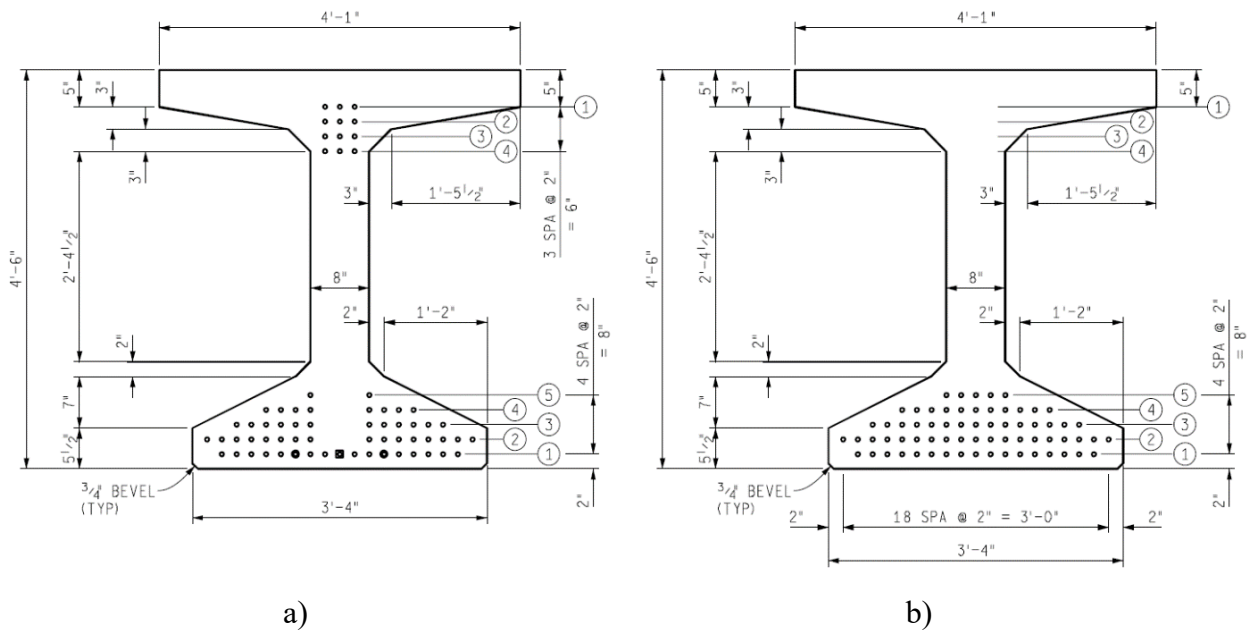


Figure E.1. PCBT 54 girder dimension and strand pattern at; a) end and b) midspan

The strand patterns at ends and midspan are shown in Figure E.1. The strands indicated inside the circles and squares represent debonded strands whose information is provided in Table E.1.

Table E.1. Debonded strand information

STRAND LEGEND & DEBONDING TABLE		
LEGEND	NUMBER	LENGTH
■	1	4'-0"
●	2	12'-0"

Additionally, the detailed strand location as well as debonded strand information is provided in Table E.2. All the harped strands are harped at $0.33L_G$ with respect to end of the girder.

Table E.2. Detailed strand location information

Strand Location Table																		
Location	End (a)									Midspan (b)								
	Bottom					Top				Bottom					Top			
Layer #	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>	<u>5</u>	<u>1</u>	<u>2</u>	<u>3</u>	<u>4</u>
Strand #	17	16	12	8	2	3	3	3	3	17	19	15	11	5	0	0	0	0

Furthermore, the strand properties are provided in Table E.3.

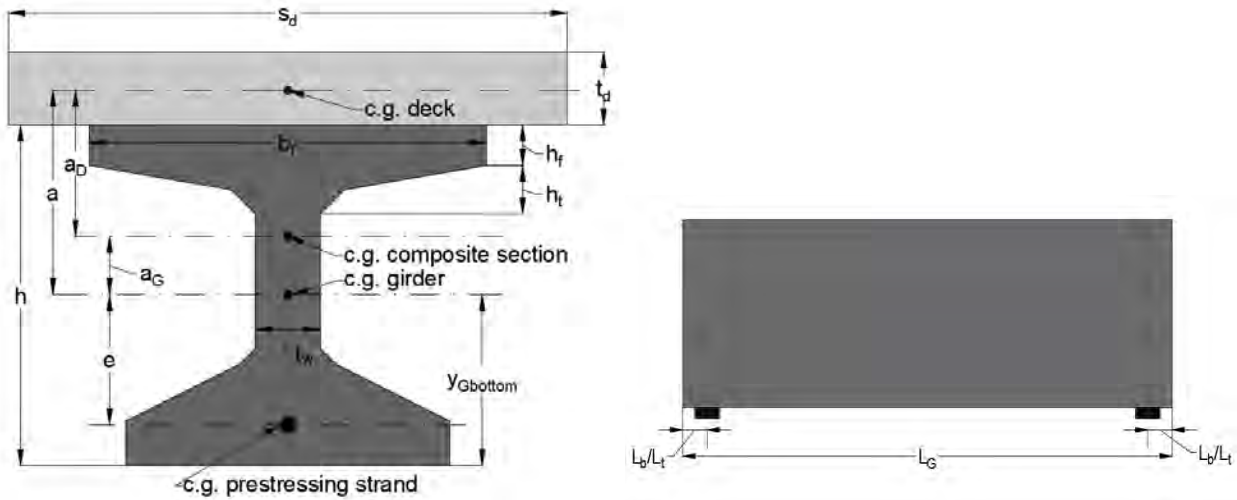
Table E.3. Prestressing strand properties

Parameter	Value
Modulus of elasticity (E_{ps}) (ksi)	28500
Nominal diameter (d_{ps}) (in.)	0.6
Ultimate tensile strength (f_{pu}) (ksi)	270
Jacking stress (f_{pj}) (ksi)	202.5
Yield stress (f_{py}) (ksi)	243
Strand type	Low-relaxation

The girder used in this project is a type of bulb tee girder with 49" top flange width and 54" height, as shown in Figure E.1. Girder properties and other relevant information are given in Table E.4. In addition to this, Figure E.2 indicates the notation used in this example problem.

Table E.4. Parameters for PCBT 54" girder and project specific information

Parameter	Value
Gross Area (A_G) ($in.^2$)	1022.3
Centroid of girder measured from bottom ($y_{Gbottom}$) ($in.$)	27
Gross moment of inertia (I_G) ($in.^4$)	412056
Girder height (h) ($in.$)	54
Perimeter of the cross-section (S_G) ($in.$)	251.4
Top flange width (b_f) ($in.$)	49
Top flange thickness (h_f) ($in.$)	5
Web thickness (t_w) ($in.$)	8
Height of top inclined surface (h_t) ($in.$)	3.51
Unit-weight of cross-section* (γ_G) (kcf)	0.155
Girder length (L_G) ($ft.$)	136.9
Girder overhang length at precast facility (L_t) ($in.$)	21.5
Girder overhang length at bridge site (L_b) ($in.$)	5.5
Average spacing of girder from center to center [†] (s_d) ($ft.$)	8.41
Thermal expansion coefficient (α) ($1/^\circ F$)	6.7E-06
*Calculated based on AASHTO LRFD (2020) Bridge Design Specifications Table 3.5.1.1 by Reinforcement weight is considered as 5 pcf.	
[†] For the fascia beams, spacing should be adjusted accordingly.	

**Figure E.2.** Notations used for bridge super-structure

The initial and 28-days specified concrete compressive strength for the girder are $f'_{ci} = 7$ ksi and $f'_c = 10$ ksi, respectively. Moreover, the deck concrete compressive strength at 28-days is given $f'_{cd} = 4$ ksi. The unit weight of concrete is 0.145 kcf. It is assumed that the deck is replaced

after the bridge has been in service for 50 years. The properties related to first and second decks are given in Table E.5.

Table E.5. Parameters related to first and second decks

Parameter for the First Deck	Value
Unit-weight of deck and haunch* (γ_{d1}) (kcf)	0.150
Deck thickness (t_{d1}) (in.)	9
Average haunch thickness (t_{h1}) (in.)	0
Overlay thickness (t_{o1}) (in.)	2
Unit-weight of overlay (γ_{o1}) (kcf)	0.140
Barrier load (w_{b1}) (klf)	0.475
Parameter for the Second Deck	Value
Unit-weight of deck and haunch* (γ_{d2}) (kcf)	0.150
Deck thickness (t_{d2}) (in.)	9
Average haunch thickness (t_{h2}) (in.)	0
Overlay thickness (t_{o2}) (in.)	2
Unit-weight of overlay (γ_{o2}) (kcf)	0.140
Barrier load (w_{b2}) (klf)	0.475
*Reinforcement weight is considered as 5 pcf.	

In this example, the following time arrangement is going to be utilized. It should be noted that the barrier and overlay load is applied to super-structure simultaneously so that there is no additional time-dependent activity taking place between these two events.

Table E.6. Time arrangement for major activities

Event	time index (t)	$t(t)$ (days)
Detensioning of strands	<u>1</u>	1
Time at which the deck is placed	<u>2</u>	28
Time at placement of barriers and the opening of bridge to traffic	<u>3</u>	35
Time at placement of overlay	<u>4</u>	35
Time at deck removal for projects that features a deck replacement	<u>5</u>	18250
Time at new deck placement	<u>6</u>	18260
Time at placement of barrier and reopening of bridge to traffic	<u>7</u>	18267
Time at placement of overlay	<u>8</u>	18267
Time at the end of service life	<u>9</u>	27375

For this analysis, ACI 209R-92 model is going to be used with back-calculated parameters for the development of modulus of elasticity with time. In addition, AASHTO LRFD (2020) equation is going to be used for the estimation of modulus of elasticity at release and 28-days. Throughout the analysis, specified concrete compressive strength values are going to be utilized with no overstrength factor. Moreover, for conducting time-dependent analysis, AASHTO LRFD (2020) Body creep and shrinkage models are going to be used assuming that flexural and tensile creep are equal to compression creep. In addition to this, three sections are going to be considered along the half-span to minimize the computation effort. To estimate the deflection from curvature diagram, Simpson's rule is employed for numerical integration. It is assumed that the ambient relative humidity is 70 % for Michigan based on the map provided in AASHTO LRFD (2020) 9th Edition Figure 5.4.2.3.3-1. Moreover, it is assumed that both decks (first and second) are cured 7-days using moist curing, and the girder is cured 1-day using steam curing. While generating the time matrix for the analysis, only 1 time-step is going to be taken between the major activities stated above. Finally, the sign convention adopted in this example problem is given in the Figure E.3.

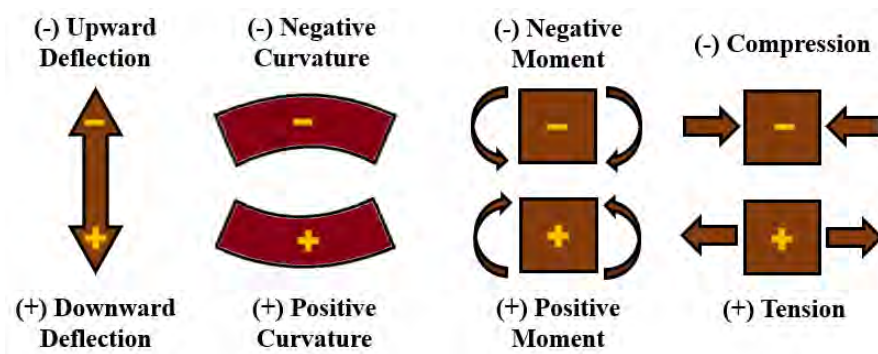


Figure E.3. Sign convention

E.2. Definition of Parameters

- *Generate the time and section matrix*

Since only 1 time step is going to be considered in this example, the time matrix generated for this specific example would be same as in Table E.7. In addition to this, since three sections are going to be considered along the half-span, the section matrix can be assigned as follows:

Table E.7. Section array

<i>section index (x)</i>	<i>loc(x) (in.)</i>
<u>1</u>	0
<u>2</u>	410.7
<u>3</u>	821.4

It should be noted that when creating time and section dependent arrays, index number is going to be used for simplicity throughout the analysis. For example, say a parameter $A(t, x)$ is both time and section-dependent. $A(6, 3)$ refers to the value of A at time $t(t) = t(6) = 18260$ days and $loc(x) = loc(3) = 821.4$ in. from girder end.

- *Determine the prestressing related parameters such as eccentricity, strand area, debonding length, transfer length along the span and jacking force*
- Transfer length:

As specified in AASHTO LRFD (2020) 9th Edition section 5.9.4.3.1, the transfer length is assumed to be 60 times strand diameter. Between the points where bonding commences and transfer length finishes, the stress in the strands is assumed to be varying linearly.

$$L_{tr} = 60 \cdot d_{ps} \quad \text{E-1}$$

$$L_{tr} = 60 \cdot (0.6) = 36 \text{ in.}$$

- Average debonding length, total debonded strand area and eccentricity of debonded strands:

Using Table E.1, the average debonding length, total debonded strand area and eccentricity of debonded strands can be calculated as follows:

$$L_{dbavg} = \frac{\sum L_{dbi} \cdot N_i}{N_{db}} \quad \text{E-2}$$

$$A_{psdb} = A_{psi} \cdot N_{db} \quad \text{E-3}$$

$$e_{db} = y_{Gbottom} - \frac{\sum y_{dbi} \cdot N_i}{N_{db}} \quad \text{E-4}$$

Nominal area for Ø0.6 in. strand:

$$A_{psi} = 0.217 \text{ in.}^2$$

$$L_{dbavg} = \frac{(4 \cdot 12) \cdot 1 + (12 \cdot 12) \cdot 2}{1 + 2} = 112 \text{ in.}$$

$$A_{psdb} = (0.217) \cdot (2 + 1) = 0.651 \text{ in.}^2$$

$$e_{db} = 27 - \frac{(2) \cdot 2 + (2) \cdot 1}{1 + 2} = 25 \text{ in.}$$

- Average harping length, total harped strand area and eccentricity of harped strands:

Similarly, using Figure E.1 and Table E.2, the average harping length, total harped strand area and eccentricity of harped strands along the half-span can be calculated as follows:

$$L_{hpavg} = \frac{\sum L_{hpi} \cdot N_i}{N_{hp}} \quad \text{E-5}$$

$$A_{pshp} = A_{psi} \cdot N_{hp} \quad \text{E-6}$$

$$e_{hp,end} = y_{Gbottom} - \frac{\sum y_{hpi,end} \cdot N_i}{N_{hp}} \quad \text{E-7}$$

$$e_{hp,mid} = y_{Gbottom} - \frac{\sum y_{hpi,mid} \cdot N_i}{N_{hp}} \quad \text{E-8}$$

Since they change along the span, eccentricity of harped strands should be determined for each section shown in Table E.7. For this, a linear equation is going to be generated such that the eccentricity of harped strand along the span can be determined as follows:

$$\text{if } x \leq L_{hpavg} \quad e_{hp}(x) = e_{hp,end} - \frac{x \cdot (e_{hp,end} - e_{hp,mid})}{L_{hpavg}} \quad \text{E-9}$$

$$\text{if } x > L_{hpavg} \quad e_{hp}(x) = e_{hp,mid} \quad \text{E-10}$$

Since all the harped strands are harped at $0.33L_G$ from end of girder, average harping length equals to $0.33L_G$.

$$L_{hpavg} = 0.33 \cdot L_G = (0.33) \cdot (136.9 \cdot 12) = 542.1 \text{ in.}$$

$$A_{pshp} = (0.217) \cdot (3 + 3 + 3 + 3) = 2.604 \text{ in.}^2$$

$$e_{hp,end} = 27 - \frac{(3 \cdot 43 + 3 \cdot 45 + 3 \cdot 47 + 3 \cdot 51)}{(3 + 3 + 3 + 3)} = -19.5 \text{ in.}$$

$$e_{hp,mid} = 27 - \frac{(3 \cdot 4 + 3 \cdot 6 + 3 \cdot 8 + 3 \cdot 10)}{(3 + 3 + 3 + 3)} = 20 \text{ in.}$$

$$\text{if } x \leq 542.1 \text{ in.} \quad e_{hp}(x) = -19.5 - \frac{x \cdot (-19.5 - 20)}{542.1} = x \cdot (0.0729) - 19.5$$

$$\text{if } x > 542.1 \text{ in.} \quad e_{hp}(x) = 20$$

The eccentricities of harped strands at sections given in Table E.7 are provided in Table E.8.

Table E.8. Eccentricity of harped strands along the half-span

$e_{hp}(x) \text{ (in.)}$		
$e_{hp}(1)$	$e_{hp}(2)$	$e_{hp}(3)$
-19.5	10.44	20.00

- Total area and eccentricity of fully bonded, straight strands:

Using Figure E.1 and Table E.2, the total area of the remaining fully bonded, straight strand and their eccentricity can be calculated as follows:

$$A_{psst} = A_{psi} \cdot N_{st} \quad E-11$$

$$e_{st} = y_{Gbottom} - \frac{\sum y_{sti} \cdot N_i}{N_{st}} \quad E-12$$

$$A_{psst} = (67 - 3 - 12) \cdot 0.217 = 11.284 \text{ in.}^2$$

$$e_{st} = 27 - \frac{14 \cdot 2 + 16 \cdot 4 + 12 \cdot 6 + 8 \cdot 8 + 2 \cdot 10}{14 + 16 + 12 + 8 + 2} = 22.23 \text{ in.}$$

- Total effective strand area and net eccentricity along the half-span:

Using the calculation above and Table E.2, the effective strand area and net eccentricity along the half-span can be calculated as follows:

- *Net eccentricity at end, midspan and end of debonding length*

$$e_e = \frac{e_{st} \cdot N_{st} + e_{hp,end} \cdot N_{hp}}{N_{st} + N_{hp}} \quad E-13$$

$$e_{atdb} = \frac{e_{st} \cdot N_{st} + e_{db} \cdot N_{db} + \left[e_{hp,end} - (e_{hp,end} - e_{hp,mid}) \cdot \frac{L_{dbavg}}{L_{hpavg}} \right] \cdot N_{hp}}{N_{st} + N_{hp} + N_{db}} \quad E-14$$

$$e_m = \frac{e_{st} \cdot N_{st} + e_{hp,mid} \cdot N_{hp} + e_{db} \cdot N_{db}}{N_{st} + N_{hp} + N_{db}} \quad E-15$$

$$e_e = \frac{22.23 \cdot 52 - 19.5 \cdot 12}{52 + 12} = 14.64 \text{ in.}$$

$$e_{atdb} = \frac{22.23 \cdot 52 + 25 \cdot 3 + \left[-19.5 - (-19.5 - 20) \cdot \frac{112}{542.1} \right] \cdot 12}{52 + 12 + 3} = 16.34 \text{ in.}$$

$$e_m = \frac{22.23 \cdot 52 + 20 \cdot 12 + 25 \cdot 3}{52 + 12 + 3} = 21.95 \text{ in.}$$

▪ *Net eccentricity along the half-span*

✓ if $x \leq L_{dbavg} = 112 \text{ in.} \& x \leq L_{hpavg} = 542.1 \text{ in.}$

$$e(x) = e_e + \frac{x \cdot (e_{atdb} - e_e)}{L_{dbavg}} \quad \text{E-16}$$

$$e(x) = 14.64 + \frac{x \cdot (16.34 - 14.64)}{112} = 14.64 + 0.0152 \cdot x$$

✓ if $L_{dbavg} = 112 \text{ in.} < x \leq L_{hpavg} = 542.1 \text{ in.}$

$$e(x) = e_{atdb} + \frac{(e_m - e_{atdb}) \cdot (x - L_{dbavg})}{(L_{hpavg} - L_{dbavg})} \quad \text{E-17}$$

$$e(x) = 16.34 + \frac{(21.95 - 16.34) \cdot (x - 112)}{(542.1 - 112)} = 16.34 + 0.0130 \cdot (x - 112)$$

✓ if $L_{hpavg} = 542.1 \text{ in.} < x$

$$e(x) = e_m \quad \text{E-18}$$

$$e(x) = 21.95$$

Table E.9. Net eccentricity along the half-span

$e(x) \text{ (in.)}$		
$e(1)$	$e(2)$	$e(3)$
14.64	20.22	21.95

▪ *Effective prestressing area along the half-span*

✓ if $x \leq L_{dbavg} = 112 \text{ in.}$

$$A_{ps}(x) = A_{psst} + A_{pshp} \quad \text{E-19}$$

$$A_{ps}(x) = 11.284 + 2.604 = 13.388 \text{ in.}^2$$

✓ if $x > L_{dbavg} = 112 \text{ in.}$

$$A_{ps}(x) = A_{psst} + A_{psdb} + A_{pshp} \quad \text{E-20}$$

$$A_{ps}(x) = 11.284 + 0.651 + 2.604 = 14.539 \text{ in.}^2$$

Table E.10. Effective strand area along the half-span

$A_{ps}(x) (\text{in.}^2)$		
$A_{ps}(1)$	$A_{ps}(2)$	$A_{ps}(3)$
13.388	14.539	14.539

- *Generate creep and shrinkage arrays*

- Creep function based on AASHTO LRFD (2020) 9th edition

$$\psi(t, t_i) = 1.9 \cdot k_s \cdot k_{hc} \cdot k_f \cdot k_{td} \cdot t_i^{-0.118} \quad \text{E-21}$$

$$k_s = 1.45 - 0.13 \cdot \left(\frac{V}{S}\right) \geq 1.0 \quad \text{E-22}$$

$$k_{hc} = 1.56 - 0.008 \cdot H_r \quad \text{E-23}$$

$$k_f = \frac{5}{1 + f'_{ci}} \quad \text{E-24}$$

$$k_{td}(t) = \frac{t}{12 \cdot \left(\frac{100 - 4 \cdot f'_{ci}}{f'_{ci} + 20}\right) + t} \quad \text{E-25}$$

- Shrinkage function based on AASHTO LRFD (2020) 9th edition

$$\varepsilon_{sh}(t) = k_s \cdot k_{hs} \cdot k_f \cdot k_{td} \cdot 0.00048 \quad \text{E-26}$$

$$k_{hs} = 2.00 - 0.014 \cdot H_r \quad \text{E-27}$$

Here, the volume-to-surface ratio can also be expressed as surface-to-perimeter ratio. Moreover, according to AASHTO LRFD (2020) Bridge Design Specification section 5.4.2.3.3, shrinkage as determined in Eqn. E-26 should be increased by 20 % if the concrete is exposed to drying before 5 days of curing have elapsed.

- *Creep and shrinkage calculation for girder*

$$k_s = 1.45 - 0.13 \cdot \left(\frac{1022.3}{251.4} \right) = 0.921 < 1.0 \rightarrow k_s = 1.0$$

$$k_{hc} = 1.56 - 0.008 \cdot (70) = 1.00$$

$$k_{hs} = 2.00 - 0.014 \cdot (70) = 1.02$$

$$k_f = \frac{5}{1+7} = 0.625$$

$$k_{tdc}(t) = \frac{t - t_i}{12 \cdot \left(\frac{100 - 4 \cdot (7)}{7 + 20} \right) + (t - t_i)} = \frac{t - t_i}{32 + (t - t_i)}$$

$$k_{tds}(t) = \frac{t-1}{12 \cdot \left(\frac{100 - 4 \cdot (7)}{7+20} \right) + (t-1)} = \frac{t-1}{31+t}$$

Using Eqns. E-21 and E-26, creep and shrinkage strain values at desired time can be calculated. To exemplify, $\psi(t_9, t_4) = \psi(27375, 36)$ and $\varepsilon_{sh}(t_9) = \varepsilon_{sh}(27375)$ can be calculated as shown below. In addition to this, creep and shrinkage arrays are provided in Table E.11 and Table E.12.

$$\psi(27375,36) = 1.9 \cdot 1.0 \cdot 1.0 \cdot 0.625 \cdot \frac{27375 - 36}{32 + (27375 - 36)} \cdot 36^{-0.118} = 0.78$$

$$\varepsilon_{sh}(27375) = 1.2 \cdot 1.0 \cdot 1.02 \cdot 0.625 \cdot \frac{27375 - 1}{31 + 27375} \cdot 0.00048 = 367 \mu\varepsilon$$

Table E.11. Creep function for girder

[illegible]

Table E.12. Shrinkage array for girder

Event	<i>time index (t)</i>	$\epsilon_{shG}(t) (\mu\epsilon)$
Detensioning of strands	<u>1</u>	0.00
Time at which the deck is placed	<u>2</u>	168
Time at placement of barriers and the opening of bridge to traffic	<u>3</u>	189
Time at placement of overlay	<u>4</u>	189
Time at deck removal for projects that features a deck replacement	<u>5</u>	367
Time at new deck placement	<u>6</u>	367
Time at placement of barrier and reopening of bridge to traffic	<u>7</u>	367
Time at placement of overlay	<u>8</u>	367
Time at the end of service life	<u>9</u>	367

- *Creep and shrinkage calculation for 1st deck*

Since initial concrete compressive strength for the deck is generally unknown, it is assumed:

$$f'_{cdi} = 0.80 \cdot f'_{cd} \quad \text{E-28}$$

$$f'_{cdi} = 0.80 \cdot 4 = 3.2 \text{ ksi}$$

It should be noted that since the deck is 7-days moist-cured, there will be no shrinkage that takes places within the 7-days period after it is cast. Similarly, before deck is cast, there will be no creep in the deck.

$$A_{d1} = s_d \cdot t_{d1} + t_{h1} \cdot b_f \quad \text{E-29}$$

$$S_{d1} = 2 \cdot s_d - b_f + t_{h1} \quad \text{E-30}$$

$$\frac{V_{d1}}{A_{d1}} = \frac{A_{d1}}{S_{d1}} \quad \text{E-31}$$

$$A_{d1} = (8.41 \cdot 12) \cdot 9 = 908.28 \text{ in.}^2$$

$$S_{d1} = 2 \cdot (8.41 \cdot 12) - 49 + 0 = 152.84 \text{ in.}$$

$$k_s = 1.45 - 0.13 \cdot \left(\frac{908.28}{152.84} \right) = 0.677 < 1.0 \rightarrow k_s = 1.0$$

$$k_{hc} = 1.56 - 0.008 \cdot (70) = 1.00$$

$$k_{hs} = 2.00 - 0.014 \cdot (70) = 1.02$$

$$k_f = \frac{5}{1 + 3.2} = 1.190$$

$$k_{tdc}(t) = \frac{t - t_i}{12 \cdot \left(\frac{100 - 4 \cdot (3.2)}{3.2 + 20} \right) + (t - t_i)} = \frac{t - t_i}{45.1 + (t - t_i)}$$

$$k_{tds}(t) = \frac{t - 29 - 7}{12 \cdot \left(\frac{100 - 4 \cdot (3.2)}{3.2 + 20} \right) + (t - 29 - 7)} = \frac{t - 36}{9.10 + t}$$

Using Eqns. E-21 and E-26, the creep and shrinkage array for the 1st deck can be obtained as follows:

Table E.13. Creep function for 1st deck

$\psi_{D1}(t, t_i)$		t (days)					
		<u>29</u>	<u>36</u>	<u>18250</u>	<u>18260</u>	<u>18627</u>	<u>27375</u>
t_i (days)	<u>29</u>	0.00	0.00	0.00	0.00	0.00	0.00
	<u>36</u>	-	0.00	1.79	1.79	1.79	1.79
	<u>18250</u>	-	-	0.00	0.13	0.20	0.71
	<u>18260</u>	-	-	-	0.00	0.10	0.71
	<u>18267</u>	-	-	-	-	0.00	0.71
	<u>27375</u>	-	-	-	-	-	0.00

t_i = loading time, t = desired time

Table E.14. Shrinkage array for 1st deck

Event	<i>time index (t)</i>	$\epsilon_{shD1}(t) (\mu\epsilon)$
Detensioning of strands	<u>1</u>	0
Time at which the deck is placed	<u>2</u>	0
Time at placement of barriers and the opening of bridge to traffic	<u>3</u>	0
Time at placement of overlay	<u>4</u>	0
Time at deck removal for projects that features a deck replacement	<u>5</u>	581
Time at new deck placement	<u>6</u>	581
Time at placement of barrier and reopening of bridge to traffic	<u>7</u>	581
Time at placement of overlay	<u>8</u>	581
Time at the end of service life	<u>9</u>	582

▪ *Creep and shrinkage calculation for 2nd deck*

Similar to the 1st deck, the creep and shrinkage arrays for the 2nd deck can also be determined. For simplicity, it is assumed that the 2nd deck has exactly the same features with 1st deck. The only formulation that is different for the 2nd deck is as follows:

$$k_{tds}(t) = \frac{t - 18260 - 7}{12 \cdot \left(\frac{100 - 4 \cdot (3.2)}{3.2 + 20} \right) + (t - 18260 - 7)} = \frac{t - 18267}{t - 18221.9}$$

Table E.15. Creep function for 2nd deck

$\psi_{D2}(t, t_i)$		<i>t (days)</i>		
		<u>18260</u>	<u>18627</u>	<u>27375</u>
<i>t_i (days)</i>	<u>18260</u>	0.00	0.00	0.00
	<u>18267</u>	-	0.00	1.79
	<u>27375</u>	-	-	0.00
<i>t_i</i> = loading time, <i>t</i> = desired time				

Table E.16. Shrinkage array for 2nd deck

Event	<i>time index (t)</i>	<i>ε_{shD2} (t) (με)</i>
Detensioning of strands	<u>1</u>	0
Time at which the deck is placed	<u>2</u>	0
Time at placement of barriers and the opening of bridge to traffic	<u>3</u>	0
Time at placement of overlay	<u>4</u>	0
Time at deck removal for projects that features a deck replacement	<u>5</u>	0
Time at new deck placement	<u>6</u>	0
Time at reopening of bridge to traffic	<u>7</u>	0
Time at placement of overlay	<u>8</u>	0
Time at the end of service life	<u>9</u>	580

- *Generate time and section dependent arrays for material and section properties*
- Modulus of elasticity based on AASHTO LRFD (2020) 9th edition with ACI 209R-92 development function

$$E_c(t) = \left(\frac{t}{\alpha + \beta t} \right)^{0.5} E_{c28} \quad \text{E-32}$$

$$E_{ci} = 120,000 K_1 w^2 (f'_{ci})^{1/3} \quad \text{E-33}$$

$$E_{c28} = 120,000 K_1 w^2 (f'_c)^{1/3} \quad \text{E-34}$$

- *For girder*

For girder, development model parameters, α and β , are back-calculated using the equations below:

$$\beta = \frac{E_{c28}^2 \cdot t_1 - 28 E_{ci}^2}{E_{ci}^2 \cdot (t_1 - 28)} \quad \text{E-35}$$

$$\alpha = 28(1 - \beta) \quad \text{E-36}$$

According to AASHTO LRFD (2020) Bridge Design Specifications Table 3.5.1.1, the unit weight of concrete, w , may be estimated in terms of kcf using the equations below:

$$\text{If } f'_c > 4 \text{ ksi} \quad w = 0.140 + 0.001 \cdot (f'_c) \quad \text{E-37}$$

$$\text{If } f'_c \leq 4 \text{ ksi} \quad w = 0.145 \text{ kcf} \quad \text{E-38}$$

$$w = 0.140 + 0.001 \cdot (10) = 0.150 \text{ kcf}$$

$$E'_{ci} = 120,000 \cdot (1.0) \cdot (0.150)^2 \cdot (7)^{1/3} = 5165 \text{ ksi}$$

$$E'_{c28} = 120,000 \cdot (1.0) \cdot (0.150)^2 \cdot (10)^{1/3} = 5817 \text{ ksi}$$

$$\beta = \frac{(5817)^2 \cdot (1) - 28 \cdot (5165)^2}{(5165)^2 \cdot (1 - 28)} = 0.990$$

$$\alpha = 28 \cdot (1 - 0.990) = 0.278$$

$$E_{cG}(t) = \left(\frac{t}{0.278 + (0.990) \cdot t} \right)^{0.5} \cdot 5817$$

Table E.17. Elastic modulus development for girder with time

Event	time index (t)	$E_{cG}(t)$ (ksi)
Detensioning of strands	<u>1</u>	5165
Time at which the deck is placed	<u>2</u>	5817
Time at placement of barriers and the opening of bridge to traffic	<u>3</u>	5823
Time at placement of overlay	<u>4</u>	5823
Time at deck removal for projects that features a deck replacement	<u>5</u>	5846
Time at new deck placement	<u>6</u>	5846
Time at placement of barrier and reopening of bridge to traffic	<u>7</u>	5846
Time at placement of overlay	<u>8</u>	5846
Time at the end of service life	<u>9</u>	5846

- For 1st and 2nd deck

Unlike in girder concrete, in deck concrete, same development model with coefficients corresponding to Type I – Moist-cured concrete is going to be utilized.

$$\alpha = 4 \text{ \& } \beta = 0.85 \text{ \& } w = 0.145 \text{ kcf}$$

$$E'_{c28} = 120,000 \cdot (1.0) \cdot (0.145)^2 \cdot (4)^{1/3} = 3644.1 \text{ ksi}$$

$$E_{cD1}(t) = \left(\frac{t - 28}{4 + (0.85) \cdot (t - 28)} \right)^{0.5} \cdot 3644.1$$

$$E_{cD2}(t) = \left(\frac{t - 18260}{4 + (0.85) \cdot (t - 18260)} \right)^{0.5} \cdot 3644.1$$

Table E.18. Elastic modulus development for 1st and 2nd deck with time

Event	<i>time index (t)</i>	$E_{cD1}(t)$ (ksi)	$E_{cD2}(t)$ (ksi)
Detensioning of strands	<u>1</u>	-	-
Time at which the deck is placed	<u>2</u>	-	-
Time at placement of barriers and the opening of bridge to traffic	<u>3</u>	3447	-
Time at placement of overlay	<u>4</u>	3447	-
Time at deck removal for projects that features a deck replacement	<u>5</u>	4343	-
Time at new deck placement	<u>6</u>	4343	-
Time at placement of barrier and reopening of bridge to traffic	<u>7</u>	4343	3359
Time at placement of overlay	<u>8</u>	4343	3359
Time at the end of service life	<u>9</u>	4344	4343

- Modular ratio and transformed area

Since transformed section properties are utilized in Approach No. 6, all the materials apart from girder are transformed to girder by using the equation below. Since girder modulus of elasticity varies with time, the modular ratio for the elements will also vary with respect to time.

$$\eta(t) = \frac{E_{tr}}{E_{cG}(t)} \quad \text{E-39}$$

$$A_{tr} = A \cdot \eta(t) \quad \text{E-40}$$

- *For prestressing strands*

$$\eta_{ps}(t) = \frac{E_{ps}}{E_{cG}(t)} = \frac{28500}{E_{cG}(t)}$$

$$A_{pstr}(t, x) = A_{ps}(x) \cdot (\eta_{ps}(t) - 1)$$

Table E.19. Modular ratio and transformed area for prestressing strands

$A_{pstr}(t, x) (in.^2)$		$\eta_{ps}(t)$	<i>section index (x)</i>		
			<u>1</u>	<u>2</u>	<u>3</u>
<i>time index (t)</i>	<u>1</u>	5.52	62.7	65.7	65.7
	<u>2</u>	4.90	54.2	56.7	56.7
	<u>3</u>	4.89	54.1	56.7	56.7
	<u>4</u>	4.89	54.1	56.7	56.7
	<u>5</u>	4.88	53.8	56.3	56.3
	<u>6</u>	4.88	53.8	56.3	56.3
	<u>7</u>	4.88	53.8	56.3	56.3
	<u>8</u>	4.88	53.8	56.3	56.3
	<u>9</u>	4.88	53.8	56.3	56.3

- For 1st and 2nd deck

$$\eta_D(t) = \frac{E_{cD}(t)}{E_{cG}(t)}$$

$$A_{dtr}(t) = A_d \cdot \eta_d(t)$$

Table E.20. Modular ratio and transformed area for the 1st and 2nd deck

<i>time index (t)</i>	Modular ratio		Transformed deck area (in.²)	
	$\eta_{d1}(t)$	$\eta_{d2}(t)$	$A_{d1tr}(t)$	$A_{d2tr}(t)$
<u>3</u>	0.61	-	555.6	-
<u>4</u>	0.61	-	555.6	-
<u>5</u>	0.72	-	656.8	-
<u>6</u>	0.72	-	656.8	-
<u>7</u>	0.72	0.61	656.8	553.4
<u>8</u>	0.72	0.61	656.8	553.4
<u>9</u>	0.72	0.72	656.8	656.8

- Transformed area for composite and non-composite structures

$$A_{nctr}(t, x) = A_G + A_{pstr}(t, x)$$

$$A_{ctr}(t, x) = A_G + A_{pstr}(t, x) + A_{dtr}(t)$$

Table E.21. Net transformed area for the non-composite structure

$A_{nctr}(t, x) (in.^2)$		<i>section index</i> (x)		
		<u>1</u>	<u>2</u>	<u>3</u>
<i>time index</i> (t)	<u>1</u>	1085	1088	1088
	<u>2</u>	1076	1079	1079
	<u>3</u>	1076	1079	1079
	<u>4</u>	1076	1079	1079
	<u>5</u>	1076	1079	1079
	<u>6</u>	1076	1079	1079
	<u>7</u>	1076	1079	1079
	<u>8</u>	1076	1079	1079
	<u>9</u>	1076	1079	1079

Table E.22. Net transformed area for the 1st and 2nd composite structures

$A_{ctr}(t, x) (in.^2)$		$A_{c1tr}(t, x)$			$A_{c2tr}(t, x)$		
		<i>section index</i> (x)			<i>section index</i> (x)		
		<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<i>time index</i> (t)	<u>3</u>	1632	1635	1635	-	-	-
	<u>4</u>	1733	1736	1736	-	-	-
	<u>5</u>	1733	1736	1736	-	-	-
	<u>6</u>	1733	1736	1736	-	-	-
	<u>7</u>	1733	1736	1736	1630	1632	1632
	<u>8</u>	1733	1736	1736	1733	1735	1735
	<u>9</u>	1733	1736	1736	1733	1735	1735

- *Transformed moment of inertia for non-composite structure*

$$y_{psbottom}(x) = y_{Gbottom} - e(x) \quad E-41$$

$$y_t(t, x) = \frac{A_G \cdot y_{Gbottom} + A_{pstr}(t, x) \cdot y_{psbottom}(x)}{A_{nctr}(t, x)} \quad E-42$$

$$e_{ts}(t, x) = y_t(t, x) - y_{psbottom}(x) \quad E-43$$

$$I_{ts}(t, x) = I_G + A_G \cdot (y_{Gbottom} - y_t(t, x))^2 + A_{pstr}(t, x) \cdot (y_t(t, x) - y_{psbottom}(x))^2 \quad E-44$$

Table E.23. Location of prestressing strands with respect to bottom of girder

$y_{psbottom}(x)$ (in.)		
$y_{psbottom}(1)$	$y_{psbottom}(2)$	$y_{psbottom}(3)$
12.5	6.7	5.0

Table E.24. Eccentricity of prestressing strand in transformed non-composite structure

$e_{ts}(t, x)$ (in.)		<i>section index</i> (x)		
		<u>1</u>	<u>2</u>	<u>3</u>
<i>time index</i> (t)	<u>1</u>	13.7	19.1	20.6
	<u>2</u>	13.8	19.2	20.8
	<u>3</u>	13.8	19.2	20.8
	<u>4</u>	13.8	19.2	20.8
	<u>5</u>	13.8	19.3	20.8
	<u>6</u>	13.8	19.3	20.8
	<u>7</u>	13.8	19.3	20.8
	<u>8</u>	13.8	19.3	20.8
	<u>9</u>	13.8	19.3	20.8

Table E.25. Centroid of transformed non-composite structure with respect to bottom of girder

$y_t(t, x)$ (in.)		<i>section index</i> (x)		
		<u>1</u>	<u>2</u>	<u>3</u>
<i>time index</i> (t)	<u>1</u>	26.2	25.8	25.7
	<u>2</u>	26.3	25.9	25.8
	<u>3</u>	26.3	25.9	25.9
	<u>4</u>	26.3	25.9	25.9
	<u>5</u>	26.3	25.9	25.9
	<u>6</u>	26.3	25.9	25.9
	<u>7</u>	26.3	25.9	25.9
	<u>8</u>	26.3	25.9	25.9
	<u>9</u>	26.3	25.9	25.9

Table E.26. Moment of inertia of transformed non-composite structure

$I_{ts}(t, x) (in.^4)$		<i>section index</i> (x)		
		<u>1</u>	<u>2</u>	<u>3</u>
time index (t)	<u>1</u>	424490	437520	441810
	<u>2</u>	422870	434210	437950
	<u>3</u>	422860	434190	437920
	<u>4</u>	422860	434190	437920
	<u>5</u>	422810	434080	437800
	<u>6</u>	422810	434080	437800
	<u>7</u>	422810	434080	437800
	<u>8</u>	422810	434080	437800
	<u>9</u>	422810	434080	437790

- *Transformed moment of inertia for composite structure*

$$y_{dc} = \frac{s_d \cdot t_d \cdot (0.5 \cdot t_d + t_h) + 0.5 \cdot b_f \cdot t_h^2}{A_d} \quad E-45$$

$$y_{dbottom} = h + y_{dc} \quad E-46$$

$$y_c(t, x) = \frac{A_G \cdot y_{Gbottom} + A_{pstr}(t, x) \cdot y_{psbottom}(x) + A_{dtr} \cdot y_{dbottom}}{A_{ctr}(t, x)} \quad E-47$$

$$e_c(t, x) = y_c(t, x) - y_{psbottom}(x) \quad E-48$$

$$I_d = \frac{s_d \cdot t_d^3 + b_f \cdot t_h^3}{12} + s_d \cdot t_d \cdot (y_{dc} - 0.5 \cdot t_d - t_h)^2 + b_f \cdot t_h \cdot (y_{dc} - 0.5 \cdot t_h)^2 \quad E-49$$

$$I_{dtr} = I_d \cdot \eta_{d1}(t) \quad E-50$$

$$I_c(t, x) = I_G + A_G \cdot (y_c(t, x) - y_{Gbottom})^2 + A_{pstr}(t, x) \cdot (y_c(t, x) - y_{psbottom}(x))^2 + I_{dtr}(t) + A_{dtr}(t) \cdot (y_c(t, x) - y_{dbottom})^2 \quad E-51$$

Table E.27. Centroid of transformed composite structure with respect to bottom of girder

$y_c(t, x)$ (in.)		$y_{c1}(t, x)$			$y_{c2}(t, x)$		
		<i>section index</i> (x)			<i>section index</i> (x)		
		<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<i>time index</i> (t)	<u>3</u>	37.2	37.0	36.9	-	-	-
	<u>4</u>	38.5	37.0	36.9	-	-	-
	<u>5</u>	38.5	37.0	38.2	-	-	-
	<u>6</u>	38.5	38.3	38.2	-	-	-
	<u>7</u>	38.5	38.3	38.2	37.2	37.0	36.9
	<u>8</u>	38.5	38.3	38.2	37.2	37.0	36.9
	<u>9</u>	38.5	38.3	38.2	38.5	38.3	38.2

Table 2.22. Eccentricity of prestressing strand in transformed composite structure

$e_c(t, x)$ (in.)		$e_{c1}(t, x)$			$e_{c2}(t, x)$		
		<i>section index</i> (x)			<i>section index</i> (x)		
		<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<i>time index</i> (t)	<u>3</u>	24.7	30.3	31.9	-	-	-
	<u>4</u>	24.7	30.3	31.9	-	-	-
	<u>5</u>	26.0	31.6	33.2	-	-	-
	<u>6</u>	26.0	31.6	33.2	-	-	-
	<u>7</u>	26.0	31.6	33.2	24.7	30.3	31.9
	<u>8</u>	26.0	31.6	33.2	24.7	30.3	31.9
	<u>9</u>	26.0	31.6	33.2	26.0	31.6	33.2

Table E.28. Transformed moment of inertia for the 1st and 2nd deck

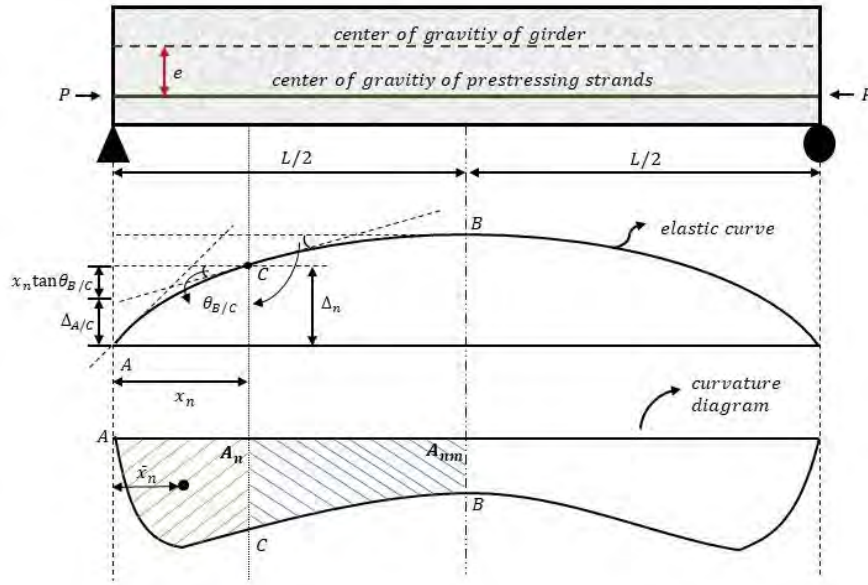
<i>time index</i> (t)	Transformed moment of inertia (in. ⁴)	
	I_{d1tr}	I_{d2tr}
<u>3</u>	3751	-
<u>4</u>	3751	-
<u>5</u>	4434	-
<u>6</u>	4434	-
<u>7</u>	4434	3736
<u>8</u>	4434	3736
<u>9</u>	4434	4433

Table E.29. Moment of inertia of transformed composite structure with 1st and 2nd deck

$I_c(t, x) \text{ (in.}^4\text{)}$		$I_{c1}(t, x)$			$I_{c2}(t, x)$		
		section index (x)			section index (x)		
		<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
time index (t)	<u>3</u>	807250	826900	832690	-	-	-
	<u>4</u>	807250	826900	832690	-	-	-
	<u>5</u>	850800	871330	877330	-	-	-
	<u>6</u>	850800	871330	877330	-	-	-
	<u>7</u>	850800	871330	877330	806060	825600	831360
	<u>8</u>	850800	871330	877330	806060	825600	831360
	<u>9</u>	850810	871340	877340	850780	871310	877310

- Simpson's 1/3rd rule for numerical integration

A general idea behind the Simpson's 1/3rd rule is explained briefly below.

**Figure E.4.** Elastic curve and curvature diagram for typical prestressed concrete beam at release

$$\Delta_n = \Delta_{A/C} + x_n \tan \theta_{B/C} \quad \text{E-52}$$

$$\theta_{B/C} = A_{nm} \quad \text{E-53}$$

$$\Delta_{A/C} = A_n \bar{x}_n \quad \text{E-54}$$

$$\bar{x}_n = \frac{Q_n}{A_n} \quad \text{E-55}$$

$$Q_n = \frac{\Delta x}{3} (\phi_1 x_1 + 4\phi_2 x_2 + 2\phi_3 x_3 + 4\phi_4 x_4 + 2\phi_5 x_5 + \cdots + 4\phi_{n-1} x_{n-1} + \phi_n x_n) \quad \text{E-56}$$

$$A_n = \frac{\Delta x}{3} (\phi_1 + 4\phi_2 + 2\phi_3 + 4\phi_4 + 2\phi_5 + \cdots + 4\phi_{n-1} + \phi_n) \quad \text{E-57}$$

$$A_{nm} = \frac{\Delta x}{3} (\phi_n + 4\phi_{n+1} + 2\phi_{n+2} + \cdots + 4\phi_{m-1} + \phi_m) \quad \text{E-58}$$

$$\Delta x = \frac{L_G/2}{\text{number of sections} - 1} \quad \text{E-59}$$

- Time-dependent deflection calculations

Note: For both instantaneous and time-dependent deflection calculations, all matrices such as location, load effects, material properties and creep and shrinkage functions will be recalled using index number.

$$K_i(t, x) = \frac{1}{1 + \frac{E_{ps} \cdot A_{ps}(x)}{E_{cG}(1) \cdot A_G} \cdot \left(1 + \frac{A_G \cdot e(x)^2}{I_G}\right) \cdot [1 + \psi_G(t, t_1)]} \quad \text{E-60}$$

$$f_{psj}(x) = -\frac{(N_{psdb} + N_{psbp} + N_{psst})}{A_{ps}(x)} \quad \text{E-61}$$

$$z = \text{rownumber}(N_G(t, x)) \quad \text{E-62}$$

$$q = \text{rownumber}(N_D(t, x)) \quad \text{E-63}$$

$$m = \text{rownumber}(\text{loc}(x)) \quad \text{E-64}$$

$$\emptyset_p = \emptyset_0 \quad \text{E-65}$$

$$N_{tloss}(1, loc(x)) = N_{tloss_0} \quad E-66$$

$$\Delta f_{pR}(1, loc(x)) = \Delta f_{pR_0} \quad E-67$$

$$\Delta \emptyset_t(1, loc(x)) = 0 \quad E-68$$

- For non-composite structure

$$i = index(t_i) \quad E-69$$

$$p = index(t_{i+1}) \quad E-70$$

$$r = i \quad E-71$$

$$k = [r; (r + 1); (r + 2); \dots (p - 2); (p - 1)] \quad E-72$$

- For each value in "k" array:

$$Co_{NG}(1, loc(x)) = 0 \ \& \ Co_{MG}(1, loc(x)) = 0 \quad E-73$$

$$\varepsilon_{shG} = -(\varepsilon_G(k + 1) - \varepsilon_G(k)) \quad E-74$$

$$u = [1; 2; \dots (z - 1); z] \quad E-75$$

- For each value in "u" array:

$$\psi_{crG} = \psi_G(t(k + 1), t(u)) - \psi_G(t(k), t(u)) \quad E-76$$

$$\psi_{crG, flex} = \psi_{crG} \cdot \beta_{flex} \quad E-77$$

$$E_G = E_{cG}(u) \quad E-78$$

$$h = [1; 2; \dots (m - 1); m] \quad E-79$$

- For each value in "h" array:

$$\text{if } N_G(u, h) > 0 \quad \psi_{crG,ax} = \psi_{crG} \cdot \beta_{ax} \quad \text{E-80}$$

$$\text{if } N_G(u, h) \leq 0 \quad \psi_{crG,ax} = \psi_{crG} \quad \text{E-81}$$

$$Co_{NG}(h) = \frac{N_G(u, h) \cdot \psi_{crG,ax}}{E_G \cdot A_G} + Co_{NG} \quad \text{E-82}$$

$$Co_{MG}(h) = \frac{M_G(u, h) \cdot \psi_{crG,flex}}{E_G \cdot I_G} + Co_{MG} \quad \text{E-83}$$

- End of "h" array.
- End of "u" array.

$$\psi_{crG,ax} = \psi_{crG} \quad \text{E-84}$$

$$stopper = 1 \quad \text{E-85}$$

$$j = [1; 2; \dots (m - 1); m] \quad \text{E-86}$$

➤ For each value in "j" array:

$$M_{co} = \begin{bmatrix} 1 & 1 & 0 & 0 & 0 & 0 \\ 0 & e(j) & 1 & 0 & 0 & 0 \\ 0 & 0 & -\frac{1 + \psi_{crG,flex}}{E_G \cdot I_G} & 0 & 0 & 1 \\ 0 & -\frac{1}{E_{ps} \cdot A_{ps}(j)} & 0 & 0 & 1 & 0 \\ -\frac{1 + \psi_{crG,ax}}{E_G \cdot A_G} & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & -1 & 1 & -e(j) \end{bmatrix} \quad \text{E-87}$$

$$M_c = \begin{bmatrix} 0 \\ 0 \\ Co_{MG}(j) \\ 0 \\ Co_{NG}(j) + \varepsilon_{shG} \\ 0 \end{bmatrix} \quad \text{E-88}$$

$$M_s = [M_{co}^{-1}] \cdot [M_c] \quad \text{E-89}$$

stopper = 0

if $M_s(1) > 0$

$$\psi_{crG,ax} = \psi_{crG} \cdot \beta_{ax} \quad \text{E-90}$$

if *stopper* = 1

continue

$$f_{pR}(k, j) = \Delta f_{pR}(k, j) \quad \text{E-91}$$

$$\Delta f_{pR}(k+1, j) = - \left[\frac{(f_{psj}(j) + \Delta f_{pES}(j)) \cdot \log(24 \cdot t(k) + 1)}{K'_L \cdot \log(24 \cdot t_1 + 1)} \right. \\ \text{if } f_{psj}(j) \\ \geq 0.55 \cdot f_{py} \cdot \left(\frac{f_{psj}(j) + \Delta f_{pES}(j)}{f_{py}} - 0.55 \right) \left. \right] \left[1 \right. \\ - \Delta f_{pES}(j) \\ \left. + \frac{3 \cdot (N_{tloss}(k, j) + M_s(2))}{(f_{psj}(j) + \Delta f_{pES}(j)) \cdot A_{ps}(j)} \right] \cdot K_i(k+1, j) \cdot A_{ps}(j) \quad \text{E-92}$$

$$\text{if } f_{psj}(j) \\ < 0.55 \cdot f_{py} - \Delta f_{pES}(j) \quad \Delta f_{pR}(k+1, j) = 0 \quad \text{E-93}$$

$$\text{if } \Delta f_{pR}(k+1, j) \\ > \Delta f_{pR}(k, j) \quad \Delta f_{pR}(k+1, j) = \Delta f_{pR}(k, j) \quad \text{E-94}$$

$$f_{pR}(k+1, j) = \Delta f_{pR}(k+1, j) - f_{pR}(k, j) \quad \text{E-95}$$

$$M_{pR}(k+1, j) = f_{pR}(k+1, j) \cdot [e(j) - (y_{Gbottom} - y_t(k, x))] \quad \text{E-96}$$

$$N_{tloss}(k+1, j) = M_s(2) + N_{tloss}(k, j) \quad \text{E-97}$$

$$N_G(k+1, j) = M_s(1) - f_{pR}(k+1, j) \cdot \frac{A_G}{A_{nctr}(k, j)} \\ - \frac{M_{pR}(k+1, j) \cdot A_G \cdot (y_t(k, x) - y_{Gbottom})}{I_{ts}(k, x)} \quad \text{E-98}$$

$$M_G(k+1, j) = M_s(3) - M_{pR}(k+1, j) \cdot \frac{I_G}{I_{ts}(k, j)} \quad \text{E-99}$$

$$\Delta\phi_t(k+1, j) = M_s(6) + \Delta\phi_t(k, j) - \frac{M_{pR}(k+1, j)}{E_G \cdot I_{ts}(k, j)} \quad \text{E-100}$$

$$stopper = 1 \quad \text{E-101}$$

$$\psi_{crG, ax} = \psi_{crG} \quad \text{E-102}$$

➤ End of "j" array.

$$\phi_t(k+1, loc(x)) = \phi_p + \Delta\phi_t(k+1, loc(x)) \quad \text{E-103}$$

➤ End of "k" array.

▪ For composite structure

$$y_{dG} = y_{dbottom} - y_{Gbottom} \quad \text{E-104}$$

$$i = index(t_i) \quad \text{E-105}$$

$$p = index(t_{i+1}) \quad \text{E-106}$$

$$r = i \quad \text{E-107}$$

$$g = index(t_2) \quad \text{E-108}$$

$$k = [r; (r+1); (r+2); \dots (p-2); (p-1)] \quad \text{E-109}$$

• For each value in "k" array:

$$Co_{NG}(1, loc(x)) = 0 \ \& \ Co_{MG}(1, loc(x)) = 0 \quad \text{E-110}$$

$$Co_{ND}(1, loc(x)) = 0 \ \& \ Co_{MD}(1, loc(x)) = 0$$

$$\varepsilon_{shG} = -(\varepsilon_G(k+1) - \varepsilon_G(k)) \quad \text{E-111}$$

$$\varepsilon_{shD} = -(\varepsilon_{d1}(k+1) - \varepsilon_{d1}(k))$$

$$u = [1; 2; \dots (z - 1); z] \quad \text{E-112}$$

- For each value in "u" array:

$$\psi_{crG} = \psi_G(t(k + 1), t(u)) - \psi_G(t(k), t(u)) \quad \text{E-113}$$

$$\psi_{crG,flex} = \psi_{crG} \cdot \beta_{flex} \quad \text{E-114}$$

$$E_G = E_{cG}(u) \quad \text{E-115}$$

$$h = [1; 2; \dots (m - 1); m] \quad \text{E-116}$$

- For each value in "h" array:

$$\text{if } N_G(u, h) > 0 \quad \psi_{crG,ax} = \psi_{crG} \cdot \beta_{ax} \quad \text{E-117}$$

$$\text{if } N_G(u, h) \leq 0 \quad \psi_{crG,ax} = \psi_{crG} \quad \text{E-118}$$

$$Co_{NG}(h) = \frac{N_G(u, h) \cdot \psi_{crG,ax}}{E_G \cdot A_G} + Co_{NG} \quad \text{E-119}$$

$$Co_{MG}(h) = \frac{M_G(u, h) \cdot \psi_{crG,flex}}{E_G \cdot I_G} + Co_{MG} \quad \text{E-120}$$

- End of "h" array.
- End of "u" array.

$$z = [g; g + 1; \dots (q - 1); q] \quad \text{E-121}$$

- For each value in "z" array.

$$\psi_{crD} = \psi_D(t(k + 1), t(z)) - \psi_D(t(k), t(z)) \quad \text{E-122}$$

$$\psi_{crD,flex} = \psi_{crD} \cdot \beta_{flex} \quad \text{E-123}$$

$$E_D = E_{cD}(z) \quad \text{E-124}$$

$$h = [1; 2; \dots (m - 1); m] \quad \text{E-125}$$

- For each value in "h" array:

$$\text{if } N_D(z, h) > 0 \quad \psi_{crD,ax} = \psi_{crD} \cdot \beta_{ax} \quad \text{E-126}$$

$$\text{if } N_D(z, h) \leq 0 \quad \psi_{crD,ax} = \psi_{crD} \quad \text{E-127}$$

$$Co_{ND}(h) = \frac{N_D(z, h) \cdot \psi_{crD,ax}}{E_D \cdot A_D} + Co_{ND} \quad \text{E-128}$$

$$Co_{MD}(h) = \frac{M_D(z, h) \cdot \psi_{crD,flex}}{E_D \cdot I_D} + Co_{MD} \quad \text{E-129}$$

- End of "h" array.
- End of "z" array.

$$\psi_{crG,ax} = \psi_{crG} \quad \text{E-130}$$

$$\psi_{crD,ax} = \psi_{crD} \quad \text{E-131}$$

$$stopper = 1 \quad \text{E-132}$$

$$j = [1; 2; \dots (m - 1); m] \quad \text{E-133}$$

- For each value in "j" array: E-134

M_{co}

$$= \begin{bmatrix} 1 & 1 & 1 & 0 & 0 & 0 & 0 & 0 & 0 \\ -y_{dG} & 0 & e(j) & 1 & 1 & 0 & 0 & 0 & 0 \\ -\frac{1 + \psi_{crD,ax}}{E_D \cdot A_D} & 0 & 0 & 0 & 0 & 1 & 0 & 0 & 0 \\ 0 & -\frac{1 + \psi_{crG,ax}}{E_G \cdot A_G} & 0 & 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & -\frac{1 + \psi_{crD,flex}}{E_D \cdot I_D} & 0 & 0 & 0 & 0 & 1 \\ 0 & 0 & 0 & 0 & -\frac{1 + \psi_{crG,flex}}{E_G \cdot I_G} & 0 & 0 & 0 & 1 \\ 0 & 0 & -\frac{1}{E_{ps} \cdot A_{ps}(j)} & 0 & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 & 0 & 1 & -1 & 0 & y_{dG} \\ 0 & 0 & 0 & 0 & 0 & 0 & -1 & 1 & -e(j) \end{bmatrix}$$

$$M_c = \begin{bmatrix} 0 \\ 0 \\ Co_{ND}(j) + \varepsilon_{shD} \\ Co_{NG}(j) + \varepsilon_{shG} \\ Co_{MD}(j) \\ Co_{MG}(j) \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

E-135

$$M_s = [M_{co}^{-1}] \cdot [M_c]$$

E-136

stopper = 0

if [$M_s(1)$ or $M_s(2)$] > 0

if $M_s(2) > 0$; $\psi_{crG,ax} = \psi_{crG} \cdot \beta_{ax}$

E-137

if stopper = 1

if $M_s(1) > 0$; $\psi_{crD,ax} = \psi_{crD} \cdot \beta_{ax}$

continue

$$f_{pR}(k, j) = \Delta f_{pR}(k, j)$$

E-138

$$\begin{aligned}
& \Delta f_{pR}(k+1, j) = - \left[\frac{(f_{psj}(j) + \Delta f_{pES}(j)) \cdot \log(24 \cdot t(k) + 1)}{K'_L \cdot \log(24 \cdot t_1 + 1)} \right. \\
& \text{if } f_{psj}(j) \\
& \geq 0.55 \cdot f_{py} \quad \cdot \left(\frac{f_{psj}(j) + \Delta f_{pES}(j)}{f_{py}} - 0.55 \right) \left. \right] \left[1 \right. \\
& - \Delta f_{pES}(j) \\
& \quad \left. + \frac{3 \cdot (N_{tloss}(k, j) + M_s(3))}{(f_{psj}(j) + \Delta f_{pES}(j)) \cdot A_{ps}(j)} \right] \cdot K_i(k+1, j) \cdot A_{ps}(j)
\end{aligned} \tag{E-139}$$

$$\begin{aligned}
& \text{if } f_{psj}(j) \\
& < 0.55 \cdot f_{py} - \Delta f_{pES}(j) \quad \Delta f_{pR}(k+1, j) = 0
\end{aligned} \tag{E-140}$$

$$\begin{aligned}
& \text{if } \Delta f_{pR}(k+1, j) \\
& > \Delta f_{pR}(k, j) \quad \Delta f_{pR}(k+1, j) = \Delta f_{pR}(k, j)
\end{aligned} \tag{E-141}$$

$$f_{pR}(k+1, j) = \Delta f_{pR}(k+1, j) - f_{pR}(k, j) \tag{E-142}$$

$$M_{pR}(k+1, j) = f_{pR}(k+1, j) \cdot e_c(j) \tag{E-143}$$

$$N_{tloss}(k+1, j) = M_s(3) + N_{tloss}(k, j) \tag{E-144}$$

$$\begin{aligned}
& \delta N_{GpR}(k+1, j) \\
& = f_{pR}(k+1, j) \cdot \frac{A_G}{A_{ctr}(k, t)}
\end{aligned} \tag{E-145}$$

$$+ \frac{M_{pR}(k+1, j) \cdot A_G \cdot (y_c(k, x) - y_{Gbottom})}{I_c(k, x)}$$

$$\begin{aligned}
& \delta N_{DpR}(k+1, j) \\
& = f_{pR}(k+1, j) \frac{A_{dtr}(k)}{A_{ctr}(k, j)} \\
& + \frac{M_{pR}(k+1, j) \cdot A_{dtr}(k) \cdot (y_c(k, x) - y_{dbottom})}{I_c(k, x)}
\end{aligned} \tag{E-146}$$

$$\delta M_{GpR}(k+1, j) = \frac{M_{pR}(k+1, j) \cdot I_G}{I_c(k, x)} \quad \text{E-147}$$

$$\delta M_{DpR}(k+1, j) = \frac{M_{pR}(k+1, j) \cdot I_{dtr}(k)}{I_c(k, x)} \quad \text{E-148}$$

$$N_G(k+1, j) = M_s(2) - \delta N_{GpR}(k+1) \quad \text{E-149}$$

$$M_G(k+1, j) = M_s(5) - \delta M_{GpR}(k+1) \quad \text{E-150}$$

$$N_D(k+1, j) = M_s(1) - \delta N_{DpR}(k+1) \quad \text{E-151}$$

$$M_D(k+1, j) = M_s(4) - \delta M_{DpR}(k+1) \quad \text{E-152}$$

$$\Delta \phi_t(k+1, j) = M_s(9) + \Delta \phi_t(k, j) - \frac{M_{pR}(k+1, j)}{E_G \cdot I_c(k, j)} \quad \text{E-153}$$

$$stopper = 1 \quad \text{E-154}$$

$$\psi_{crG, ax} = \psi_{crG} \quad \text{E-155}$$

$$\psi_{crd, ax} = \psi_{crD} \quad \text{E-156}$$

➤ End of "j" array.

$$\phi_t(k+1, loc(x)) = \phi_p + \Delta \phi_t(k+1, loc(x)) \quad \text{E-157}$$

➤ End of "k" array.

E.3. Camber at Release

➤ Girder Self-weight Effect

$$w_G = \gamma_G \cdot A_G \quad \text{E-158}$$

$$\text{if } x \leq L_t \quad M_{Gsw}(x) = -\frac{w_G \cdot x^2}{2} \quad \text{E-159}$$

$$\text{if } L_t < x \quad M_{Gsw}(x) = \frac{w_G \cdot L_G \cdot (x - L_t)}{2} - \frac{w_G \cdot x^2}{2} \quad \text{E-160}$$

$$w_G = \frac{0.155 \cdot 1022.3}{12^3} = 0.0917 \text{ kips/in.}$$

$$\text{if } x \leq 21.5 \text{ in.} \quad M_{Gsw}(x) = -\frac{0.0917 \cdot x^2}{2} = -0.046 \cdot x^2$$

$$\text{if } 21.5 < x \quad M_{Gsw}(x) = \frac{0.092 \cdot 136.9 \cdot 12 \cdot (x - 21.5)}{2} - \frac{0.092 \cdot x^2}{2}$$

Table E.30. Moment applied on non-composite structure due to girder self-weight

$M_{Gsw}(x) \text{ (kips} \cdot \text{in.)}$		
$M_{Gsw}(1)$	$M_{Gsw}(2)$	$M_{Gsw}(3)$
0	21582	29315

➤ Prestressing Effect

- Straight, fully bonded strands

$$P_{psst} = A_{psst} \cdot f_{pj} \quad \text{E-161}$$

$$\text{if } x \leq L_{tr} \quad N_{psst}(x) = -\frac{P_{psst} \cdot x}{L_{tr}} \quad \text{E-162}$$

$$\text{if } L_{tr} < x \quad N_{psst}(x) = -P_{psst} \quad \text{E-163}$$

$$M_{psst}(x) = N_{psst}(x) \cdot [e_{st} - (y_{Gbottom} - y_t(1, x))] \quad \text{E-164}$$

$$P_{psst} = 11.284 \cdot 202.5 = 2285.01 \text{ kips}$$

$$\text{if } x \leq 36 \quad N_{psst}(x) = -\frac{2285.01 \cdot x}{36} = -63.47 \cdot x$$

$$M_{psst}(x) = -63.47 \cdot x \cdot [22.23 - (27 - y_t(1, x))]$$

$$\text{if } 36 < x \quad N_{psst}(x) = -P_{psst} = -2285.01 \text{ kips}$$

$$M_{psst}(x) = -2285.01 \cdot [22.23 - (27 - y_t(1, x))]$$

Table E.31. Axial force and moment on non-composite structure due to straight strands

$N_{psst}(x) \text{ (kips)}$			$M_{psst}(x) \text{ (kips} \cdot \text{in.)}$		
$N_{psst}(1)$	$N_{psst}(2)$	$N_{psst}(3)$	$M_{psst}(1)$	$M_{psst}(2)$	$M_{psst}(3)$
0	-2285	-2285	0	-47996	-47769

- Debonded strands

$$P_{psdb} = A_{psdb} \cdot f_{pj} \quad \text{E-165}$$

$$\text{if } x \leq L_{dbavg} \quad N_{psdb}(x) = 0 \quad \text{E-166}$$

$$\text{if } L_{dbavg} < x \leq (L_{dbavg} + L_{tr}) \quad N_{psdb}(x) = -\frac{P_{psdb} \cdot (x - L_{dbavg})}{L_{tr}} \quad \text{E-167}$$

$$\text{if } (L_{dbavg} + L_{tr}) < x \quad N_{psdb}(x) = -P_{psdb} \quad \text{E-168}$$

$$M_{psdb}(x) = N_{psdb}(x) \cdot [e_{db} - (y_{Gbottom} - y_t(1, x))] \quad \text{E-169}$$

$$P_{psdb} = 0.651 \cdot 202.5 = 131.83 \text{ kips}$$

$$\text{if } x \leq 112 \quad N_{psdb}(x) = 0$$

$$M_{psdb}(x) = 0$$

$$\text{if } 112 < x \quad N_{psdb}(x) = -\frac{131.83 \cdot (x - 112)}{36} = -3.66 \cdot (x - 112)$$

$$\leq (112 + 36) \quad M_{psdb}(x) = -3.66 \cdot (x - 112) \cdot [25 - (27 - y_t(1, x))]$$

$$\text{if } (112 + 36) < x \quad N_{psdb}(x) = -131.83 \text{ kips}$$

$$M_{psdb}(x) = -131.83 \cdot [25 - (27 - y_t(1, x))]$$

Table E.32. Axial force and moment on non-composite structure due to debonded strands

$N_{psdb}(x) \text{ (kips)}$			$M_{psdb}(x) \text{ (kips} \cdot \text{in.)}$		
$N_{psdb}(1)$	$N_{psdb}(2)$	$N_{psdb}(3)$	$M_{psdb}(1)$	$M_{psdb}(2)$	$M_{psdb}(3)$
0	-1312	-132	0	-3134	-3121

- Harped strands

$$P_{pshp} = A_{pshp} \cdot f_{pj} \quad \text{E-170}$$

$$\text{if } x \leq L_{tr} \quad N_{pshp}(x) = -\frac{P_{pshp} \cdot x}{L_{tr}} \quad \text{E-171}$$

$$\text{if } L_{tr} < x \leq L_{hpavg} \quad N_{pshp}(x) = -P_{pshp} \quad \text{E-172}$$

$$\text{if } L_{hpavg} < x \quad N_{pshp}(x) = -P_{pshp} \quad \text{E-173}$$

$$M_{pshp}(x) = N_{pshp}(x) \cdot [e_{hp}(x) - (y_{Gbottom} - y_t(1, x))] \quad \text{E-174}$$

$$P_{pshp} = 2.604 \cdot 202.5 = 527.31 \text{ kips}$$

$$\text{if } x \leq 36 \quad N_{pshp}(x) = -\frac{527.31 \cdot x}{36} = -14.65 \cdot x$$

$$M_{pshp}(x) = (-14.65 \cdot x) \cdot [x \cdot (0.0729) - 19.5 - (27 - y_t(1, x))]$$

$$\text{if } 36 < x \leq 542.1 \quad N_{pshp}(x) = -527.31$$

$$M_{pshp}(x) = -527.31 \cdot [x \cdot (0.0729) - (27 - y_t(1, x))]$$

$$\text{if } 542.1 < x \quad N_{pshp}(x) = -527.31$$

$$M_{pshp}(x) = -527.31 \cdot [20 - (27 - y_t(1, x))]$$

Table E.33. Axial force and moment on non-composite structure due to harped strands

$N_{pshp}(x) \text{ (kips)}$			$M_{pshp}(x) \text{ (kips-in.)}$		
$N_{pshp}(1)$	$N_{pshp}(2)$	$N_{pshp}(3)$	$M_{pshp}(1)$	$M_{pshp}(2)$	$M_{pshp}(3)$
0	-527	-527	0	-5056	-9847

➤ Net prestressing effect

$$M_{ps}(x) = M_{psst}(x) + M_{psdb}(x) + M_{pshp}(x) \quad \text{E-175}$$

$$N_{ps}(x) = N_{psst}(x) + N_{psdb}(x) + N_{pshp}(x) \quad \text{E-176}$$

Table E.34. Axial force and moment on non-composite structure due to prestressing

$N_{ps}(x)$ (kips)			$M_{ps}(x)$ (kips – in.)		
$N_{ps}(1)$	$N_{ps}(2)$	$N_{ps}(3)$	$M_{ps}(1)$	$M_{ps}(2)$	$M_{ps}(3)$
0	-2944	-2944	0	-56186	-60737

➤ Calculation of camber at release

$$M_{Gi}(x) = M_{Gsw}(x) + M_{ps}(x) \quad \text{E-177}$$

$$N_{Gi}(x) = N_{ps}(x) \quad \text{E-178}$$

$$\phi_t(1, x) = \frac{M_{Gi}(x)}{E_{cG}(1) \cdot I_{ts}(1, x)} \quad \text{E-179}$$

Table E.35. Net axial force and moment on non-composite structure due to prestressing and girder self-weight effect

$N_{Gi}(x)$ (kips)			$M_{Gi}(x)$ (kips – in.)		
$N_{Gi}(1)$	$N_{Gi}(2)$	$N_{Gi}(3)$	$M_{Gi}(1)$	$M_{Gi}(2)$	$M_{Gi}(3)$
0	-2944	-2944	0	-34604	-31422

Table E.36. Net curvature due to combined effects at camber at release

$\phi_t(t, x)$ (1/in.)		
$\phi_t(1,1)$	$\phi_t(1,2)$	$\phi_t(1,3)$
0	-1.53E-05	-1.38E-05

Using the Eqns. between E-52 and E-59, camber at considered sections can be calculated as follows:

$$\Delta x = \frac{\frac{136.9 \cdot 12}{2}}{3 - 1} = 410.70 \text{ in.}$$

$$A_{nm}(1) = \frac{410.70}{3} \cdot [0 + 4 \cdot (-1.53 \cdot 10^{-5}) + (-1.38 \cdot 10^{-5})] = -10.27 \cdot 10^{-3} \text{ in.}^2$$

$$A_{nm}(2) = \frac{410.70}{3} \cdot [(-1.53 \cdot 10^{-5}) + (-1.38 \cdot 10^{-5})] = -3.98 \cdot 10^{-3} \text{ in.}^2$$

$$A_{nm}(3) = 0$$

$$A_n(1) = 0$$

$$A_n(2) = \frac{410.70}{3} \cdot [0 + (-1.53 \cdot 10^{-5})] = -2.09 \cdot 10^{-3} \text{ in.}^2$$

$$A_n(3) = \frac{410.70}{3} \cdot [0 + 4 \cdot (-1.53 \cdot 10^{-5}) + (-1.38 \cdot 10^{-5})] - 10.27 \cdot 10^{-3} \text{ in.}^2$$

$$Q_n(1) = 0$$

$$Q_n(2) = \frac{410.70}{3} \cdot [0 + (-1.53 \cdot 10^{-5}) \cdot 410.70] = -0.86 \text{ in.}^3$$

$$Q_n(3) = \frac{410.70}{3} \cdot [0 + 4 \cdot (-1.53 \cdot 10^{-5}) \cdot 410.70 + (-1.38 \cdot 10^{-5}) \cdot 820.40] = -4.99 \text{ in.}^3$$

Table E.37. Application of Simpson's rule and camber at release

Parameters	Section index (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$Q_n(x) \text{ (in.}^3\text{)}$	0	-0.86	-4.99
$A_n(x) \text{ (in.}^2\text{)}$	0	-2.09E-03	-10.27E-03
$\bar{x}_n(x) \text{ (in.)}$	0	411.48	485.88
$\Delta_{A/C}(x) \text{ (in.)}$	0	-0.86	-4.99
$\theta_{B/C}(x) \text{ (rad.)}$	-10.27E-03	-3.98E-03	0
$x_n(x) \text{ (in.)}$	0	410.70	820.40
$\Delta_{ni}(x) \text{ (in.)}$	0	-2.50	-4.99

E.4. Pre-erection Camber

- Elastic shortening loss

A section-dependent elastic shortening loss is calculated based on AASHTO LRFD (2020)

9th Edition section C5.9.3.2.3a-1.

$$\Delta f_{pES}(x) = - \frac{-N_{ps}(x) \cdot [I_G + e(x)^2 \cdot A_G] - e(x) \cdot M_{GSW}(x) \cdot A_G}{A_{ps}(x) \cdot [I_G + e(x)^2 \cdot A_G] + A_G \cdot I_G \cdot \frac{E_{cG}(1)}{E_{ps}}} \quad \text{E-180}$$

Table E.38. Elastic shortening loss along the half-span

$\Delta f_{pES}(x) \text{ (ksi)}$			$\Delta P_{pES}(x) \text{ (kips)}$		
$\Delta f_{pES}(1)$	$\Delta f_{pES}(2)$	$\Delta f_{pES}(3)$	$\Delta P_{pES}(1)$	$\Delta P_{pES}(2)$	$\Delta P_{pES}(3)$
0	-23	-22	0	-330	-326

- Effect of self-weight on girder only

$$M_{Gsi}(x) = \frac{M_{Gsw}(x) \cdot I_G}{I_{ts}(1, x)} \quad \text{E-181}$$

$$N_{Gsi}(x) = \frac{M_{Gsw}(x) \cdot A_G \cdot (y_t(1, x) - y_{Gbottom})}{I_{ts}(1, x)} \quad \text{E-182}$$

Table E.39. Forces on girder alone due to girder self-weight

$N_{Gsi}(x) \text{ (kips)}$			$M_{Gsi}(x) \text{ (kips-in.)}$		
$N_{Gsi}(1)$	$N_{Gsi}(2)$	$N_{Gsi}(3)$	$M_{Gsi}(1)$	$M_{Gsi}(2)$	$M_{Gsi}(3)$
0	-62	-90	0	20326	27341

- Effect of prestressing on girder only

$$M_{Gps}(x) = \frac{M_{ps}(x) \cdot I_G}{I_{ts}(1, x)} \quad \text{E-183}$$

$$N_{Gps}(x) = \frac{N_{ps}(x) \cdot A_G}{A_{nctr}(1, x)} + \frac{M_{ps}(x) \cdot A_G \cdot (y_t(1, x) - y_{Gbottom})}{I_{ts}(1, x)} \quad \text{E-184}$$

Table E.40. Forces on girder alone due to prestressing

$N_{Gps}(x) \text{ (kips)}$			$M_{Gps}(x) \text{ (kips-in.)}$		
$N_{Gps}(1)$	$N_{Gps}(2)$	$N_{Gps}(3)$	$M_{Gps}(1)$	$M_{Gps}(2)$	$M_{Gps}(3)$
0	-2615	-2593	0	-52916	-56647

- Net forces on girder due to combined effects

$$M_{Gp}(x) = M_{Gsi}(x) + M_{Gps}(x) \quad \text{E-185}$$

$$N_{Gp}(x) = N_{Gsi}(x) + N_{Gps}(x) \quad \text{E-186}$$

Table E.41. Net axial force and moment on girder due to combined effect at release

$N_{Gp}(x)$ (kips)			$M_{Gp}(x)$ (kips – in.)		
$N_{Gp}(1)$	$N_{Gp}(2)$	$N_{Gp}(3)$	$M_{Gp}(1)$	$M_{Gp}(2)$	$M_{Gp}(3)$
0	-2677	-2683	0	-32590	-29306

- Calculation of pre-erection camber

$$E_{cG}(1) = 5165 \text{ ksi}$$

$$\psi_G(t_2, t_1) - \psi_G(t_1, t_1) = \psi_G(29, 1) - \psi_G(1, 1) = 0.54$$

$$K_i(2, x) = \frac{1}{1 + \frac{28500 \cdot A_{ps}(x)}{5165 \cdot 1022.3} \cdot \left(1 + \frac{1022.3 \cdot e(x)^2}{412056}\right) \cdot [1 + 0.54]}$$

Table E.42. Coefficient K_i as a function of location

$K_i(t, x)$		
$K_i(2, 1)$	$K_i(2, 2)$	$K_i(2, 3)$
0.887	0.887	0.887

Table E.43. Stress on the strands along the span prior to transfer

$f_{psj}(x)$ (ksi)		
$f_{psj}(1)$	$f_{psj}(2)$	$f_{psj}(3)$
0.00	202.50	202.50

Table E.44. Force history array on girder as a function of time and location

index	$N_G(t, x)$ (kips)			$M_G(t, x)$ (kips – in.)		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>1</u>	0	-2677	-2683	0	-32590	-29306

Using Eqns. between E-60 and E-103, the time-dependent calculation can be performed between time t_1 and t_2 .

$$z = 1 \text{ \& } m = 3$$

$$N_{tloss_0} = 0 \text{ \& } \Delta f_{pR_0} = 0$$

$$\emptyset_0 = \phi_t(1, x)$$

$$i = 1 \ \& \ p = 2 \ \& \ r = 1$$

$$k = [1]$$

$$-[\varepsilon_G(t_2) - \varepsilon_G(t_1)] = -168 \cdot 10^{-6}$$

$$u = [1]$$

$$\psi_G(t_2, t_1) - \psi_G(t_1, t_1) = 0.54$$

$$\psi_{crG} = \psi_{crG,flex} = \psi_{crG,ax} = 0.54$$

$$E_G = 5165$$

$$Co_{NG} = [0, -2.75 \cdot 10^{-4}, -2.76 \cdot 10^{-4}]$$

$$Co_{MG} = [0, -8.32 \cdot 10^{-6}, -7.48 \cdot 10^{-6}]$$

$$j = [1, 2, 3]$$

➤ For $j = 1$

$$M_{co} = \begin{bmatrix} 1 & 1 & 0 & 0 & 0 & 0 \\ 0 & 14.5 & 1 & 0 & 0 & 0 \\ 0 & 0 & -7.25 \cdot 10^{-10} & 0 & 0 & 1 \\ 0 & -2.53 \cdot 10^{-6} & 0 & 0 & 1 & 0 \\ -2.92 \cdot 10^{-7} & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & -1 & 1 & -14.5 \end{bmatrix}$$

$$M_c = \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \\ -1.68 \cdot 10^{-4} \\ 0 \end{bmatrix} \ \& \ M_s = \begin{bmatrix} 56.56 \\ -56.56 \\ 820.05 \\ -1.51 \cdot 10^{-4} \\ -1.42 \cdot 10^{-4} \\ 5.95 \cdot 10^{-7} \end{bmatrix}$$

$$\Delta f_{pR}(2,1) = 0$$

$$f_{pR}(2,1) = 0$$

$$M_{pR}(2,1) = 0$$

$$N_{tloss}(2,1) = -56.56$$

$$N_G(2,1) = 56.56$$

$$M_G(2,1) = 820.05$$

$$\Delta\phi_t(2,1) = 5.95 \cdot 10^{-7}$$

➤ For $j = 2$

$$M_{co} = \begin{bmatrix} 1 & 1 & 0 & 0 & 0 & 0 \\ 0 & 20.31 & 1 & 0 & 0 & 0 \\ 0 & 0 & -7.25 \cdot 10^{-10} & 0 & 0 & 1 \\ 0 & -2.41 \cdot 10^{-6} & 0 & 0 & 1 & 0 \\ -2.92 \cdot 10^{-7} & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & -1 & 1 & -20.31 \end{bmatrix}$$

$$M_c = \begin{bmatrix} 0 \\ 0 \\ -8.32 \cdot 10^{-6} \\ 0 \\ -4.43 \cdot 10^{-4} \\ 0 \end{bmatrix} \& M_s = \begin{bmatrix} 203.85 \\ -203.85 \\ 4140.2 \\ -3.84 \cdot 10^{-4} \\ -4.91 \cdot 10^{-4} \\ -5.32 \cdot 10^{-6} \end{bmatrix}$$

$$\Delta f_{pR}(2,2) = - \left[\frac{(202.5 - 23) \cdot \log(24 \cdot 29 + 1)}{45 \cdot \log(24 \cdot 1 + 1)} \cdot \left(\frac{202.5 - 23}{243} - 0.55 \right) \right] \left[1 + \frac{3 \cdot (0 - 203.85)}{((202.5 - 23) \cdot 14.54)} \right] \cdot 0.887 \cdot 14.54 = -7.50$$

$$f_{pR}(2,2) = -7.50$$

$$M_{pR}(2,2) = -7.50 \cdot [20.22 - (27 - 25.7)] = -143.15$$

$$N_{tloss}(2,2) = -203.85$$

$$N_G(2,2) = 203.85 + 7.50 \cdot \frac{1022.30}{1088} + \frac{143.2 \cdot 1022.3 \cdot (25.7 - 27)}{437520} = 210.62$$

$$M_G(2,2) = 4140.2 + 143.2 \cdot \frac{412056}{437520} = 4275$$

$$\Delta\phi_t(2,2) = -5.32 \cdot 10^{-6} + \frac{143.15}{5165 \cdot 437520} = -5.26 \cdot 10^{-6}$$

➤ For $j = 3$

$$M_{co} = \begin{bmatrix} 1 & 1 & 0 & 0 & 0 & 0 \\ 0 & 21.96 & 1 & 0 & 0 & 0 \\ 0 & 0 & -7.25 \cdot 10^{-10} & 0 & 0 & 1 \\ 0 & -2.41 \cdot 10^{-6} & 0 & 0 & 1 & 0 \\ -2.92 \cdot 10^{-7} & 0 & 0 & 1 & 0 & 0 \\ 0 & 0 & 0 & -1 & 1 & -21.96 \end{bmatrix}$$

$$M_c = \begin{bmatrix} 0 \\ 0 \\ -7.48 \cdot 10^{-6} \\ 0 \\ -4.44 \cdot 10^{-4} \\ 0 \end{bmatrix} \& M_s = \begin{bmatrix} 199.14 \\ -199.14 \\ 4372.2 \\ -3.86 \cdot 10^{-4} \\ -4.80 \cdot 10^{-4} \\ -4.31 \cdot 10^{-6} \end{bmatrix}$$

$$\Delta f_{pR}(2,3) = - \left[\frac{(202.5 - 22) \cdot \log(24 \cdot 29 + 1)}{45 \cdot \log(24 \cdot 1 + 1)} \cdot \left(\frac{202.5 - 222}{243} - 0.55 \right) \right] \left[1 + \frac{3 \cdot (0 - 199.14)}{(202.5 - 22) \cdot 14.54} \right] \cdot 0.887 \cdot 14.54 = -7.61$$

$$f_{pR}(2,3) = -7.61$$

$$M_{pR}(2,3) = -7.61 \cdot [21.95 - (27 - 25.7)] = -157$$

$$N_{tloss}(2,3) = -199.14$$

$$N_G(2,3) = 199.14 + 7.61 \cdot \frac{1022.30}{1088} + \frac{157 \cdot 1022.3 \cdot (25.7 - 27)}{441810} = 206$$

$$M_G(2,3) = 4372.2 + 157 \cdot \frac{412056.0}{441810} = 4518.6$$

$$\Delta \phi_t(2,3) = -4.31 \cdot 10^{-6} + \frac{157}{5165 \cdot 441810} = -4.24 \cdot 10^{-6}$$

Table E.45. Change in curvature between time t_1 and t_2

$\Delta \phi_t$ (1/in.)		
$\Delta \phi_t$	$\Delta \phi_t$	$\Delta \phi_t$
5.95E-07	-5.26E-06	-4.24E-06

Table E.46. Net curvature at pre-erection

$\phi_t(t, x) \text{ (1/in.)}$		
$\phi_t(2,1)$	$\phi_t(2,2)$	$\phi_t(2,3)$
5.95E-07	-2.06E-05	-1.80E-05

Using net curvature values provided in Table E.46 and Eqns. between E-52 and E-59, the pre-erection camber can be calculated, as shown in Table E.47.

Table E.47. Pre-erection camber

Camber (in.)	section index (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{np}(x)$	0	-3.33	-6.65

E.5. Camber just after Placement of 1st Deck

- Effect due to change in support condition

$$\text{if } x \leq 5.5 \quad M_{Gsp}(x) = -\frac{0.092 \cdot x^2}{2} = -0.046 \cdot x^2 \quad \text{E-187}$$

$$\text{if } 5.5 < x \quad M_{Gsp}(x) = \frac{0.092 \cdot 136.9 \cdot 12 \cdot (x - 5.5)}{2} - \frac{0.092 \cdot x^2}{2} \quad \text{E-188}$$

Table E.48. Moment applied on non-composite structure due to girder self-weight on bridge site

$M_{Gsp}(x) \text{ (kips} \cdot \text{in.)}$		
$M_{Gsp}(1)$	$M_{Gsp}(2)$	$M_{Gsp}(3)$
0	22787	30520

$$\Delta\phi_{sc}(x) = \frac{M_{Gsp}(x) - M_{Gsw}(x)}{E_{CG}(2) \cdot I_{ts}(2, x)} \quad \text{E-189}$$

Table E.49. Change in curvature due to change in support conditions

$\Delta\phi_{sc}(x) \text{ (1/in.)}$		
$\Delta\phi_{sc}(1)$	$\Delta\phi_{sc}(2)$	$\Delta\phi_{sc}(3)$
0	4.77E-07	4.73E-07

- Deck self-weight effect

$$w_{d1} = A_{d1} \cdot \gamma_{d1} = \frac{908.28 \cdot 0.150}{12^3} = 0.0788 \text{ kips/in.}$$

$$\text{if } x \leq 5.5 \quad M_{d1sw}(x) = -0.0394 \cdot x^2 \quad \text{E-190}$$

$$\text{if } 5.5 < x \quad M_{d1sw}(x) = 64.33 \cdot x - 0.0394 \cdot (x - 5.5)^2 - 352.60 \quad \text{E-191}$$

Table E.50. Moment applied on non-composite structure due to 1st deck self-weight

$M_{d1sw}(x)$ (kips · in.)		
$M_{d1sw}(1)$	$M_{d1sw}(2)$	$M_{d1sw}(3)$
0	19592	26242

$$\Delta\phi_{d1sw}(x) = \frac{M_{d1sw}(x)}{E_{CG}(2) \cdot I_{ts}(2, x)} \quad \text{E-192}$$

Table E.51. Change in curvature due to 1st deck self-weight

$\Delta\phi_{d1sw}(x)$ (1/in.)		
$\Delta\phi_{d1sw}(1)$	$\Delta\phi_{d1sw}(2)$	$\Delta\phi_{d1sw}(3)$
0	7.76E-06	1.03E-05

To obtain the net curvature just after 1st deck is placed, the net curvature values from previous time-step Table E.46 should be summed up with the change in curvature values given in Table E.49 and Table E.51.

Table E.52. Net curvature just after 1st deck is placed

$\phi_{ad1p}(x)$ (1/in.)		
$\phi_{ad1p}(1)$	$\phi_{ad1p}(2)$	$\phi_{ad1p}(3)$
5.95E-07	-1.23E-05	-7.24E-06

- Calculation of camber just after placement of 1st deck

Using eqns. between E-52 and E-59 with the net curvature values given in Table E.51, the camber just after 1st deck is placed can be estimated.

Table E.53. Camber just after placement of 1st deck

Camber (in.)	section index (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nad1p}(x)$	0	-1.79	-3.59

E.6. Camber just before Application of Barrier and Overlay Loads on 1st Superstructure

In this part, a time-dependent analysis for composite structure is conducted. To do so, the force history array for girder should be updated by considering the change in forces on the girder due to previous time-dependent effects between t_1 and t_2 , change in support conditions and deck self-weight.

Table E.54. Change in forces on girder due to time-dependent effects between t_1 and t_2

$\Delta N_{Gt_1t_2}(x)$ (kips)			$\Delta M_{Gt_1t_2}(x)$ (kips – in.)		
$\Delta N_{Gt_1t_2}(1)$	$\Delta N_{Gt_1t_2}(2)$	$\Delta N_{Gt_1t_2}(3)$	$\Delta M_{Gt_1t_2}(1)$	$\Delta M_{Gt_1t_2}(2)$	$\Delta M_{Gt_1t_2}(3)$
56	211	206	820	4275	4519

$$\Delta N_{Gsc}(x) = \frac{[M_{Gsp}(x) - M_{Gsw}(x)] \cdot A_G \cdot [y_t(2, x) - y_{Gbottom}]}{I_{ts}(2, x)} \quad \text{E-193}$$

$$\Delta M_{Gsc}(x) = \frac{[M_{Gsp}(x) - M_{Gsw}(x)] \cdot I_G}{I_{ts}(2, x)} \quad \text{E-194}$$

Table E.55. Change in forces on girder due to change in support conditions

ΔN_{Gsc} (kips)			ΔM_{Gsc} (kips – in.)		
$\Delta N_{Gsc}(1)$	$\Delta N_{Gsc}(2)$	$\Delta N_{Gsc}(3)$	$\Delta M_{Gsc}(1)$	$\Delta M_{Gsc}(2)$	$\Delta M_{Gsc}(3)$
0	-3	-3	0	1144	1134

$$\Delta N_{Gd1sw}(x) = \frac{M_{d1sw}(x) \cdot A_G \cdot [y_t(2, x) - y_{Gbottom}]}{I_{ts}(2, x)} \quad \text{E-195}$$

$$\Delta M_{Gd1sw}(x) = \frac{M_{d1sw}(x) \cdot I_G}{I_{ts}(2, x)} \quad \text{E-196}$$

Table E.56. Change in forces on girder due to 1st deck self-weight

ΔN_{Gd1sw} (kips)			ΔM_{Gd1sw} (kips – in.)		
$\Delta N_{Gd1sw}(1)$	$\Delta N_{Gd1sw}(2)$	$\Delta N_{Gd1sw}(3)$	$\Delta M_{Gd1sw}(1)$	$\Delta M_{Gd1sw}(2)$	$\Delta M_{Gd1sw}(3)$
0	-49	-71	0	18592	24690

The second row in Table E.57 can be obtained by simply summing the values up in the tables Table E.54, Table E.55 and Table E.56.

Table E.57. Net force history array just after 1st deck is placed

<i>index</i>	<i>N_G(t, x) (kips)</i>			<i>M_G(x) (kips – in.)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>1</u>	0	-2677	-2683	0	-32590	-29306
<u>2</u>	57	158	132	820	24011	30343

Using Eqns. between E-60 and E-68, and E-104 and E-157, the time-dependent calculation can be performed time t_2 and t_3 .

$$z = 2 \text{ \& } m = 3 \text{ \& } q = 1$$

$$\Delta f_{pR_0} = \begin{bmatrix} 0 & 0 & 0 \\ 0 & -7.5 & -7.6 \end{bmatrix}$$

$$N_{tloss_0} = \begin{bmatrix} 0 & 0 & 0 \\ -56.6 & -204 & -199 \end{bmatrix}$$

$$\emptyset_0 = [5.95 \cdot 10^{-7} \quad -1.23 \cdot 10^{-5} \quad -7.24 \cdot 10^{-6}]$$

$$y_{dG} = 31.5 \text{ in.}$$

$$i = 2 \text{ \& } p = 3 \text{ \& } r = 2$$

$$g = [2] \text{ \& } k = [2]$$

- For $k = 2$

$$Co_{NG}(1, loc(x)) = 0 \text{ \& } Co_{MG}(1, loc(x)) = 0$$

$$Co_{ND}(1, loc(x)) = 0 \text{ \& } Co_{MD}(1, loc(x)) = 0$$

$$\varepsilon_{shG} = -(\varepsilon_G(3) - \varepsilon_G(2)) \cdot 10^{-6} = -21.12 \cdot 10^{-6}$$

$$\varepsilon_{shD} = -(\varepsilon_{d1}(3) - \varepsilon_{d1}(2)) = 0$$

$$u = [1; 2]$$

$$\psi_G(t(3), t(1)) - \psi_G(t(2), t(1)) = 0.6203 - 0.5542 = 0.06$$

$$M_c = \begin{bmatrix} 0 \\ 0 \\ 0 \\ -1.98 \cdot 10^{-5} \\ 0 \\ 4.92 \cdot 10^{-8} \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

$$M_s = [M_{co}^{-1}] \cdot [M_c]$$

$$f_{pR}(2,1) = 0$$

$$\Delta f_{pR}(3,1) = 0$$

$$f_{pR}(3,1) = 0$$

$$M_{pR}(3,1) = 0$$

$$N_{tloss}(3,1) = -6.8 - 56.6 = -63.3$$

$$\delta N_{GpR}(3,1) = 0$$

$$\delta N_{DpR}(3,1) = 0$$

$$\delta M_{GpR}(3,1) = 0$$

$$\delta M_{DpR}(3,1) = 0$$

$$\delta N_{GpR}(3,1) = 0$$

$$N_G(3,1) = 6.8$$

$$M_G(3,1) = 97.9$$

$$N_D(3,1) = 0$$

$$M_D(3,1) = 0$$

$$\Delta \phi_t(3,1) = 9.6 \cdot 10^{-8}$$

Repeat the same procedure for other sections: $j = 2$ and $j = 3$

➤ End of "j" array.

$$\phi_t(3, loc(x)) = \phi_p + \Delta\phi_t(k + 1, loc(x))$$

$$\phi_t(3, loc(x)) = [6.9 \cdot 10^{-7} \quad -1.2 \cdot 10^{-5} \quad -6.2 \cdot 10^{-6}]$$

➤ End of "k" array.

Table E.58. Change in curvature between time t_2 and t_3

$\Delta\phi_t(x) \text{ (1/in.)}$		
$\Delta\phi_t(1)$	$\Delta\phi_t(2)$	$\Delta\phi_t(3)$
9.6E-08	6.0E-07	1.1E-06

Using the previous net curvature values given in Table E.52 and change in curvature values in Table E.58, net curvature values just before the application of barrier and overlay loads can be obtained as follows:

Table E.59. Net curvature just before application of barrier and overlay loads

$\phi_t(3, x) \text{ (1/in.)}$		
$\phi_t(3,1)$	$\phi_t(3,2)$	$\phi_t(3,3)$
6.9E-07	-1.2E-05	-6.2E-06

Using net curvature values provided in Table E.59 and Eqns. between E-52 and E-59, camber just before application of barrier and overlay loads can be calculated as shown in Table E.60.

Table E.60. Camber just before application of barrier and overlay loads on 1st superstructure

Camber (in.)	section index (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nbas1}(x)$	0	-1.67	-3.33

E.7. Camber just after Application of Barrier and Overlay Loads on 1st Superstructure

As it is indicated above, it is assumed that barrier and overlay loads are applied simultaneously on the 1st composite structure. In this part, only instantaneous changes in camber

due to application of superimposes deadloads that is barrier and overlay are calculated. In other words, no time-dependent calculation is performed.

- Barrier load effect

$$w_{b1} = 0.04 \text{ kips/in.}$$

$$\text{if } x \leq 5.5 \quad M_{b1sw}(x) = -0.02 \cdot x^2 \quad \text{E-197}$$

$$\text{if } 5.5 < x \quad M_{b1sw}(x) = 32.636 \cdot x - 0.02 \cdot (x - 5.5)^2 - 180.103 \quad \text{E-198}$$

$$\Delta\phi_{b1sw}(x) = \frac{M_{b1sw}(x)}{E_{cG}(3) \cdot I_{c1}(3, x)} \quad \text{E-199}$$

- Overlay load effect

$$w_{o1} = t_{o1} \cdot s_d \cdot \gamma_{o1} = \frac{2 \cdot 100.92 \cdot 0.140}{12^3} = 0.0164 \text{ kips/in.}$$

$$\text{if } x \leq 5.5 \quad M_{o1sw}(x) = -8.18 \cdot 10^{-3} \cdot x^2 \quad \text{E-200}$$

$$\text{if } 5.5 < x \quad M_{o1sw}(x) = 13.34 \cdot x - 8.18 \cdot 10^{-3} \cdot (x - 5.5)^2 - 73.43 \quad \text{E-201}$$

$$\Delta\phi_{o1sw}(x) = \frac{M_{o1sw}(x)}{E_{cG}(4) \cdot I_{c1}(4, x)} \quad \text{E-202}$$

Table E.61. Moment applied on composite structure due to barrier and overlay loads

$M_{b1sw}(x) \text{ (kips} \cdot \text{in.)}$			$M_{o1sw}(x) \text{ (kips} \cdot \text{in.)}$		
$M_{b1sw}(1)$	$M_{b1sw}(2)$	$M_{b1sw}(3)$	$M_{o1sw}(1)$	$M_{o1sw}(2)$	$M_{o1sw}(3)$
0	9940	13313	0	4064	5443

Table E.62. Change in curvature due to barrier and overlay load

$\Delta\phi_{b1sw}(x) \text{ (1/in.)}$			$\Delta\phi_{o1sw}(x) \text{ (1/in.)}$		
$\Delta\phi_{b1sw}(1)$	$\Delta\phi_{b1sw}(2)$	$\Delta\phi_{b1sw}(3)$	$\Delta\phi_{o1sw}(1)$	$\Delta\phi_{o1sw}(2)$	$\Delta\phi_{o1sw}(3)$
0	2.06E-06	2.75E-06	0	8.44E-07	1.12E-06

- Calculation of camber just after application of barrier and overlay loads

By summing the net curvature values from previous part, Table E.59, with the change in curvature values due to barrier and overlay loads, Table E.62, the net curvature can be obtained as

shown in Table E.63. Then, using Eqns. between E-52 and E-59, the camber just after application of barrier and overlay loads can be estimated as shown in Table E.64.

Table E.63. Net curvature just after application of barrier and overlay loads

$\phi_{naas1}(x)$ (1/in.)		
$\phi_{naas1}(1)$	$\phi_{naabo1}(2)$	$\phi_{naabo1}(3)$
6.90E-07	-8.82E-06	-2.31E-06

Table E.64. Camber just after application of barrier and overlay loads

Camber (in.)	section index (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{naas1}(x)$ (in.)	0	-1.28	-2.56

E.8. Camber just before Deck Removal Process

In this part, a time-dependent analysis for composite structure is conducted. To do so, the force history array for both girder and deck should be updated by considering the change in forces on the girder due to previous time-dependent effects between t_2 and t_3 , barrier and overlay self-weight.

- Time-dependent effect between t_2 and t_3

Table E.65. Change in forces on girder and deck due to time-dependent effects between t_2 and t_3

$\Delta N_{Gt_2t_3}$ (kips)			$\Delta M_{Gt_2t_3}$ (kips – in.)		
$\Delta N_{Gt_2t_3}(1)$	$\Delta N_{Gt_2t_3}(2)$	$\Delta N_{Gt_2t_3}(3)$	$\Delta M_{Gt_2t_3}(1)$	$\Delta M_{Gt_2t_3}(2)$	$\Delta M_{Gt_2t_3}(3)$
6.8	22.2	18.4	97.9	450.4	403.5
$\Delta N_{Dt_2t_3}$ (kips)			$\Delta M_{Dt_2t_3}$ (kips – in.)		
$\Delta N_{Dt_2t_3}(1)$	$\Delta N_{Dt_2t_3}(2)$	$\Delta N_{Dt_2t_3}(3)$	$\Delta M_{Dt_2t_3}(1)$	$\Delta M_{Dt_2t_3}(2)$	$\Delta M_{Dt_2t_3}(3)$
0	0	0	0	0	0

- Barrier load effect

$$\Delta N_{Gb1sw}(x) = \frac{M_{b1sw}(x) \cdot A_G \cdot [y_{c1}(3, x) - y_{Gbottom}]}{I_{c1}(3, x)} \quad \text{E-203}$$

$$\Delta M_{Gb1sw}(x) = \frac{M_{b1sw}(x) \cdot I_G}{I_{c1}(3, x)} \quad \text{E-204}$$

$$\Delta N_{d1b1sw}(x) = \frac{M_{b1sw}(x) \cdot A_{d1tr}(3, x) \cdot [y_{c1}(3, x) - y_{d1bottom}]}{I_{c1}(3, x)} \quad \text{E-205}$$

$$\Delta M_{d1b1sw}(x) = \frac{M_{b1sw}(x) \cdot I_{d1tr}(3, x)}{I_{c1}(3, x)} \quad \text{E-206}$$

Table E.66. Change in forces on girder and deck due to barrier load

$\Delta N_{Gb1sw}(kips)$			$\Delta M_{Gb1sw}(kips - in.)$		
$\Delta N_{Gd1sw}(1)$	$\Delta N_{Gd1sw}(2)$	$\Delta N_{Gd1sw}(3)$	$\Delta M_{Gd1sw}(1)$	$\Delta M_{Gd1sw}(2)$	$\Delta M_{Gd1sw}(3)$
0	123	163	0	4953	6588
$\Delta N_{d1b1sw}(kips)$			$\Delta M_{d1b1sw}(kips - in.)$		
$\Delta N_{d1b1sw}(1)$	$\Delta N_{d1b1sw}(2)$	$\Delta N_{d1b1sw}(3)$	$\Delta M_{d1b1sw}(1)$	$\Delta M_{d1b1sw}(2)$	$\Delta M_{d1b1sw}(3)$
0	-144	-191	0	45	60

- Overlay load effect

$$\Delta N_{Go1sw}(x) = \frac{M_{o1sw}(x) \cdot A_G \cdot [y_{c1}(4, x) - y_{Gbottom}]}{I_{c1}(4, x)} \quad \text{E-207}$$

$$\Delta M_{Go1sw}(x) = \frac{M_{b1sw}(x) \cdot I_G}{I_{c1}(4, x)} \quad \text{E-208}$$

$$\Delta N_{d1o1sw}(x) = \frac{M_{o1sw}(x) \cdot A_{d1tr}(4, x) \cdot [y_{c1}(4, x) - y_{d1bottom}]}{I_{c1}(4, x)} \quad \text{E-209}$$

$$\Delta M_{d1o1sw}(x) = \frac{M_{o1sw}(x) \cdot I_{d1tr}(4, x)}{I_{c1}(4, x)} \quad \text{E-210}$$

Table E.67. Change in forces on girder and deck due to overlay load

$\Delta N_{Go1sw}(kips)$			$\Delta M_{Go1sw}(kips - in.)$		
$\Delta N_{Go1sw}(1)$	$\Delta N_{Go1sw}(2)$	$\Delta N_{Go1sw}(3)$	$\Delta M_{Go1sw}(1)$	$\Delta M_{Go1sw}(2)$	$\Delta M_{Go1sw}(3)$
0	50	66	0	2025	2693
$\Delta N_{d1b1sw}(kips)$			$\Delta M_{d1b1sw}(kips - in.)$		
$\Delta N_{d1o1sw}(1)$	$\Delta N_{d1o1sw}(2)$	$\Delta N_{d1o1sw}(3)$	$\Delta M_{d1o1sw}(1)$	$\Delta M_{d1o1sw}(2)$	$\Delta M_{d1o1sw}(3)$
0	-59	-78	0	18	25

By summing up the change in forces for deck and girder separately due to abovementioned effects, the force history array for them can be updated as shown in Table E.68.

Table E.68. Net force history array just after application of barrier and overlay loads

<i>index</i>	$N_G(t, x)$ (<i>kips</i>)			$M_G(t, x)$ (<i>kips – in.</i>)		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>1</u>	0	-2677	-2683	0	-32590	-29306
<u>2</u>	57	158	132	820	24011	30343
<u>3</u>	7	145	181	98	5404	6992
<u>4</u>	0	50	66	0	2025	2693
<i>index</i>	$N_D(t, x)$ (<i>kips</i>)			$M_D(t, x)$ (<i>kips – in.</i>)		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>1</u>	0	0	0	0	0	0
<u>2</u>	0	0	0	0	0	0
<u>3</u>	0	-144	-191	0	45	60
<u>4</u>	0	-59	-78	0	18	25

It should be noted that although there is no time-dependent change in forces between t_3 and t_4 , an additional row is added for this in Table E.68. This is done on purpose so that the time indexes match with the *MDOTCamber* program where the effects of barrier and overlay loads are considered separately.

After obtaining the force history array for deck and girder, using Eqns. between E-60 and E-68, and E-104 and E-157, time-dependent calculation can be performed between time t_4 and t_5 . Since it is done explicitly in section E.6, it is not repeated here.

After conducting time-dependent calculation, the following change in force and curvature values can be obtained.

Table E.69. Change in forces on girder and deck due to time-dependent effects between t_4 and t_5

$\Delta N_{Gt_4t_5} (kips)$			$\Delta M_{Gt_4t_5} (kips - in.)$		
$\Delta N_{Gt_4t_5}(1)$	$\Delta N_{Gt_4t_5}(2)$	$\Delta N_{Gt_4t_5}(3)$	$\Delta M_{Gt_4t_5}(1)$	$\Delta M_{Gt_4t_5}(2)$	$\Delta M_{Gt_4t_5}(3)$
-159	6	14	7398	6649	5214
$\Delta N_{Dt_4t_5} (kips)$			$\Delta M_{Dt_4t_5} (kips - in.)$		
$\Delta N_{Dt_4t_5}(1)$	$\Delta N_{Dt_4t_5}(2)$	$\Delta N_{Dt_4t_5}(3)$	$\Delta M_{Dt_4t_5}(1)$	$\Delta M_{Dt_4t_5}(2)$	$\Delta M_{Dt_4t_5}(3)$
212	126	92	45	-1	4

Table E.70. Change in curvature between time t_4 and t_5

$\Delta \phi_t(x) (1/in.)$		
$\Delta \phi_t(1)$	$\Delta \phi_t(2)$	$\Delta \phi_t(3)$
5.74E-06	5.13E-06	7.42E-06

Using the previous net curvature values given in Table E.63 and change in curvature values in Table E.70 net curvature values just before the deck removal process can be obtained as follows:

Table E.71. Net curvature just before deck removal process

$\phi_t(5, x) (1/in.)$		
$\phi_t(5,1)$	$\phi_t(5,2)$	$\phi_t(5,3)$
6.43E-06	-3.69E-06	5.11E-06

Using net curvature values provided in Table E.71 and Eqns. between E-52 and E-59, camber just before the deck removal process can be calculated as shown in Table E.72.

Table E.72. Camber just before deck removal process

Camber (in.)	<i>section index</i> (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nbd1r}(x)$	0	-0.13	-0.26

E.9. Camber just after Deck Removal Process

Instantaneous change in camber just after deck is removed is composed of two parts: one is due to removal of self-weight loads of deck, overlay and barrier; other is due to effect of locked forces inside the deck on girder.

- Removal of barrier and overlay loads

$$\Delta N_{Gob1r}(x) = - \frac{[M_{b1sw}(x) + M_{o1sw}(x)] \cdot A_G \cdot [y_{c1}(5, x) - y_{Gbottom}]}{I_{c1}(5, x)} \quad \text{E-211}$$

$$\Delta M_{Gob1r}(x) = - \frac{[M_{b1sw}(x) + M_{o1sw}(x)] \cdot I_G}{I_{c1}(5, x)} \quad \text{E-212}$$

$$\Delta N_{d1ob1r}(x) = - \frac{[M_{b1sw}(x) + M_{o1sw}(x)] \cdot A_{d1tr}(5, x) \cdot [y_{c1}(5, x) - y_{d1bottom}]}{I_{c1}(5, x)} \quad \text{E-213}$$

$$\Delta M_{d1ob1r}(x) = - \frac{[M_{b1sw}(x) + M_{o1sw}(x)] \cdot I_{d1tr}(5, x)}{I_{c1}(5, x)} \quad \text{E-214}$$

$$\Delta \phi_{ob1r}(x) = - \frac{[M_{b1sw}(x) + M_{o1sw}(x)]}{E_{cG}(5) \cdot I_{c1}(5, x)} \quad \text{E-215}$$

Table E.73. Change in forces on girder and deck due to removal of barrier and overlay loads

$\Delta N_{Gob1r}(x)$ (kips)			$\Delta M_{Gob1r}(x)$ (kips – in.)		
$\Delta N_{Gob1r}(1)$	$\Delta N_{Gob1r}(2)$	$\Delta N_{Gob1r}(3)$	$\Delta M_{Gob1r}(1)$	$\Delta M_{Gob1r}(2)$	$\Delta M_{Gob1r}(3)$
0	-185	-244	0	-6622	-8809
$\Delta N_{d1ob1r}(x)$ (kips)			$\Delta M_{d1ob1r}(x)$ (kips – in.)		
$\Delta N_{d1ob1r}(1)$	$\Delta N_{d1ob1r}(2)$	$\Delta N_{d1ob1r}(3)$	$\Delta M_{d1ob1r}(1)$	$\Delta M_{d1ob1r}(2)$	$\Delta M_{d1ob1r}(3)$
0	214	285	0	-71	-95

Table E.74. Change in curvature due to removal of barrier and overlay loads

$\Delta \phi_{ob1r}(x)$ (1/in.)		
$\Delta \phi_{ob1r}(1)$	$\Delta \phi_{ob1r}(2)$	$\Delta \phi_{ob1r}(3)$
0	-2.75E-06	-3.66E-06

- Removal of locked forces in the deck before it is removed

Using Table E.68, Table E.69 and Table E.73, the total net forces on the deck can be calculated as shown in Table E.75.

Table E.75. Net forces on the deck just before it is removed

N_{d1br} (kips)			M_{d1br} (kips – in.)		
$N_{d1br}(1)$	$N_{d1br}(2)$	$N_{d1br}(3)$	$M_{d1br}(1)$	$M_{d1br}(2)$	$M_{d1br}(3)$
212	137	107	44	-8	-7

After determined the net forces on the deck, their effects on non-composite section is considered when estimating the camber using the equations below:

$$\Delta N_{tsd1r}(x) = N_{d1br}(x) \quad \text{E-216}$$

$$\Delta M_{tsd1r}(x) = M_{d1br}(x) + N_{d1br}(x) \cdot [y_{ts}(5, x) - y_{d1bottom}] \quad \text{E-217}$$

$$\Delta N_{Gtsd1r}(x) = \frac{\Delta N_{tsd1r}(x) \cdot A_G}{A_{nctr}(5, x)} + \frac{\Delta M_{tsd1r}(x) \cdot A_G \cdot [y_t(5, x) - y_{Gbottom}]}{I_{ts}(5, x)} \quad \text{E-218}$$

$$\Delta M_{Gtsd1r}(x) = \frac{\Delta M_{tsd1r}(x) \cdot I_G}{I_{ts}(5, x)} \quad \text{E-219}$$

$$\Delta \phi_{d1r}(x) = \frac{\Delta M_{tsd1r}(x)}{E_{cG}(5) \cdot I_{c1}(5, x)} \quad \text{E-220}$$

Table E.76. Change in forces on non-composite structure and girder due to removal of locked forces inside the deck

$\Delta N_{tsd1r}(x) \text{ (kips)}$			$\Delta M_{tsd1r}(x) \text{ (kips} - \text{in.)}$		
$\Delta N_{tsd1r}(1)$	$\Delta N_{tsd1r}(2)$	$\Delta N_{tsd1r}(3)$	$\Delta M_{tsd1r}(1)$	$\Delta M_{tsd1r}(2)$	$\Delta M_{tsd1r}(3)$
212	137	107	-6782	-4475	-3506
$\Delta N_{Gtsd1r}(x) \text{ (kips)}$			$\Delta M_{Gtsd1r}(x) \text{ (kips} - \text{in.)}$		
$\Delta N_{Gtsd1r}(1)$	$\Delta N_{Gtsd1r}(2)$	$\Delta N_{Gtsd1r}(3)$	$\Delta M_{Gtsd1r}(1)$	$\Delta M_{Gtsd1r}(2)$	$\Delta M_{Gtsd1r}(3)$
213	142	111	-6608	-4248	-3300

Table E.77. Change in curvature due to removal of locked forces inside the deck

$\Delta \phi_{d1r}(x) \text{ (1/in.)}$		
$\Delta \phi_{d1r}(1)$	$\Delta \phi_{d1r}(2)$	$\Delta \phi_{d1r}(3)$
-2.74E-06	-1.76E-06	-1.37E-06

- Removal of deck load

$$\Delta N_{Gd1swr}(x) = -\frac{M_{d1sw}(x) \cdot A_G \cdot [y_t(5, x) - y_{Gbottom}]}{I_{ts}(5, x)} \quad \text{E-221}$$

$$\Delta M_{Gd1swr}(x) = -\frac{M_{d1sw}(x) \cdot I_G}{I_{ts}(5, x)} \quad \text{E-222}$$

$$\Delta \phi_{d1swr}(x) = -\frac{M_{d1sw}(x)}{E_{cG}(5) \cdot I_{ts}(5, x)} \quad \text{E-223}$$

Table E.78. Change in forces on girder due to deck removal

$\Delta N_{Gd1swr}(x)$ (kips)			$\Delta M_{Gd1swr}(x)$ (kips – in.)		
$\Delta N_{Gd1swr}(1)$	$\Delta N_{Gd1swr}(2)$	$\Delta N_{Gd1swr}(3)$	$\Delta M_{Gd1swr}(1)$	$\Delta M_{Gd1swr}(2)$	$\Delta M_{Gd1swr}(3)$
0	49	70	0	-18598	-24699

Table E.79. Change in curvature due to deck removal

$\Delta \phi_{d1swr}(x)$ (1/in.)		
$\Delta \phi_{d1swr}(1)$	$\Delta \phi_{d1swr}(2)$	$\Delta \phi_{d1swr}(3)$
0	-7.72E-06	-1.02E-05

After considered all effects, curvature changes given in Table E.74, Table E.77 and Table E.79 need to be summed together with the previous net curvature values provided in Table E.71 so that the net curvature just after deck removal can be obtained as shown in Table E.80.

Table E.80. Net curvature just after deck removal

$\phi_{ad1r}(x)$ (1/in.)		
$\phi_{ad1r}(1)$	$\phi_{ad1r}(2)$	$\phi_{ad1r}(3)$
3.69E-06	-1.59E-05	-1.02E-05

Using net curvature values provided in Table E.80 and Eqns. between E-52 and E-59, camber just after the deck is removed can be calculated as shown in Table E.81.

Table E.81. Camber just after deck removal

Camber (in.)	section index (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nad1r}(x)$	0	-2.37	-4.73

E.10. Camber just before 2nd Deck Placement

In this part, a time-dependent analysis for non-composite structure is performed to estimate the camber just before the new deck is placed. Before starting this, force history array for girder needs to be updated by using Table E.68, Table E.69, Table E.73, Table E.76 and Table E.78.

Table E.82. Net force history array just after 1st deck is removed

<i>index</i>	$N_G(t, x)$ (kips)			$M_G(t, x)$ (kips – in.)		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>1</u>	0	-2677	-2683	0	-32590	-29306
<u>2</u>	57	158	132	820	24011	30343
<u>3</u>	7	145	181	98	5404	6992
<u>4</u>	0	50	66	0	2025	2693
<u>5</u>	55	12	-50	790	-22820	-31594

Using Eqns. between E-60 and E-103, time-dependent calculation can be performed. From analysis results, change in curvature values can be obtained as follows:

Table E.83. Change in curvature between t_5 and t_6

$\Delta\phi_t(x)$ (1/in.)		
$\Delta\phi_t(1)$	$\Delta\phi_t(2)$	$\Delta\phi_t(3)$
2.63E-08	-7.29E-07	-1.00E-07

To be able to obtain the net curvature values just before the new deck is placed, the time-dependent change in curvature values given in Table E.83 is summed up with the net curvature values from previous part presented in Table E.80. By doing this, the net curvature values just before the new deck is placed are obtained as follows:

Table E.84. Net curvature just before new deck placement

$\phi_t(6, x)$ (1/in.)		
$\phi_t(3,1)$	$\phi_t(3,2)$	$\phi_t(3,3)$
3.72E-06	-1.67E-05	-1.12E-05

Using net curvature values provided in Table E.84 and Eqns. between E-52 and E-59, camber just before new deck placed can be calculated as shown in Table E.85.

Table E.85. Camber just before new deck placement

Camber (in.)	<i>section index</i> (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nbndp}(x)$	0	-2.50	-5.00

E.11. Camber just after 2nd Deck Placement

A similar process that is explained in section F.4 is followed in this part to estimate the camber. Since it is assumed that 1st and 2nd decks are identical, equations used for 1st deck are valid for 2nd deck, however, parameters that are time-dependent should be updated accordingly.

- Deck self-weight effect

$$w_{d2} = A_{d2} \cdot \gamma_{d2} = \frac{908.28 \cdot 0.150}{12^3} = 0.0788 \text{ kips/in.}$$

$$\text{if } x \leq 5.5 \quad M_{d2sw}(x) = -0.0394 \cdot x^2 \quad \text{E-224}$$

$$\text{if } 5.5 < x \quad M_{d2sw}(x) = 64.33 \cdot x - 0.0394 \cdot (x - 5.5)^2 - 352.60 \quad \text{E-225}$$

Table E.86. Moment applied on non-composite structure due to 2nd deck self-weight

$M_{d2sw}(x) \text{ (kips} \cdot \text{in.)}$		
$M_{d2sw}(1)$	$M_{d2sw}(2)$	$M_{d2sw}(3)$
0	19592	26242

$$\Delta\phi_{d2sw}(x) = \frac{M_{d2sw}(x)}{E_{cG}(2) \cdot I_{ts}(6, x)} \quad \text{E-226}$$

Table E.87. Change in curvature due to 2nd deck self-weight

$\Delta\phi_{d2sw}(x) \text{ (1/in.)}$		
$\Delta\phi_{d2sw}(1)$	$\Delta\phi_{d2sw}(2)$	$\Delta\phi_{d2sw}(3)$
0	7.72E-06	1.03E-05

- Calculation of camber just after placement of 2nd deck

Table E.88. Net curvature just after 2nd deck is placed

$\phi_{ad2p}(x) \text{ (1/in.)}$		
$\phi_{ad2p}(1)$	$\phi_{ad2p}(2)$	$\phi_{ad2p}(3)$
3.72E-06	-8.93E-06	-9.16E-07

Using net curvature values given in Table E.88, and Eqns. between E-52 and E-59, the camber just after 2nd deck is placed can be estimated.

Table E.89. Camber just after placement of 2nd deck

Camber (in.)	<i>section index (x)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nad2p}(x)$	0	-1.06	-2.11

E.12. Camber just before Application of Barrier and Overlay Loads on 2nd Super-structure

In this part, a time-dependent analysis for composite structure is conducted similar to section F.5. To do so, the force history array for girder should be updated by considering the change in forces on the girder due to previous time-dependent effects between t_5 and t_6 and deck self-weight.

Table E.90. Change in forces on girder due to time-dependent effects between t_5 and t_6

$\Delta N_{Gt_5t_6}(x)$ (kips)			$\Delta M_{Gt_5t_6}(x)$ (kips – in.)		
$\Delta N_{Gt_5t_6}(1)$	$\Delta N_{Gt_5t_6}(2)$	$\Delta N_{Gt_5t_6}(3)$	$\Delta M_{Gt_5t_6}(1)$	$\Delta M_{Gt_5t_6}(2)$	$\Delta M_{Gt_5t_6}(3)$
0	13	18	-6	261	377

$$\Delta N_{Gd2sw}(x) = \frac{M_{d2sw}(x) \cdot A_G \cdot [y_t(6, x) - y_{Gbottom}]}{I_{ts}(6, x)} \quad \text{E-227}$$

$$\Delta M_{Gd2sw}(x) = \frac{M_{d2sw}(x) \cdot I_G}{I_{ts}(6, x)} \quad \text{E-228}$$

Table E.91. Change in forces on girder due to 2nd deck self-weight

$\Delta N_{Gd2sw}(kips)$			$\Delta M_{Gd2sw}(kips - in.)$		
$\Delta N_{Gd2sw}(1)$	$\Delta N_{Gd2sw}(2)$	$\Delta N_{Gd2sw}(3)$	$\Delta M_{Gd2sw}(1)$	$\Delta M_{Gd2sw}(2)$	$\Delta M_{Gd2sw}(3)$
0	-49	-70	0	18598	24699

The sixth row in Table E.92 can be obtained by simply summing the values up in the Table E.90 and Table E.91.

Table E.92. Net force history array just after 2nd deck is placed

<i>index</i>	<i>N_G(t,x)(kips)</i>			<i>M_G(t,x) (kips – in.)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>1</u>	0	-2677	-2683	0	-32590	-29306
<u>2</u>	57	158	132	820	24011	30343
<u>3</u>	7	145	181	98	5404	6992
<u>4</u>	0	50	66	0	2025	2693
<u>5</u>	55	12	-50	790	-22820	-31594
<u>6</u>	0	-36	-52	-6	18859	25076

Using Eqns. between E-60 and E-68, and E-104 and E-157, time-dependent calculation for 2nd composite structure between time t_6 and t_7 can be performed. Hence, the change in curvatures can be obtained as follows:

Table E.93. Change in curvature between time t_6 and t_7

$\Delta\phi_t(x) (1/in.)$		
$\Delta\phi_t(1)$	$\Delta\phi_t(2)$	$\Delta\phi_t(3)$
1.19E-08	1.32E-07	1.55E-07

Summing the change in curvature values in Table E.93 with the previous net curvature values in Table E.88, the net curvature values just before application of barrier and overlay loads on 2nd super-structure can be obtained as follows:

Table E.94. Net curvature just before application of barrier and overlay loads

$\phi_t(7,x) (1/in.)$		
$\phi_t(7,1)$	$\phi_t(7,2)$	$\phi_t(7,3)$
3.73E-06	-8.80E-06	-7.60E-07

Using net curvature values provided in Table E.94 and Eqns. between E-52 and E-59, camber just before application of barrier and overlay loads on 2nd super-structure can be calculated as follows:

Table E.95. Camber just before application of barrier and overlay loads on 2nd superstructure

Camber (in.)	<i>section array (x)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nbas2}(x)$	0	-1.03	-2.07

E.13. Camber just after Application of Barrier and Overlay Loads on 2nd Super-structure

Similarly, the process provided in section E.7, is followed in this part to estimate the camber just after application of barrier and overlay loads on 2nd super-structure. Since it is assumed that same barrier and overlay loads are applied on it, equations used previously for the effect of barrier and overlay loads on 1st super-structure are valid for 2nd super-structure, however, parameters that are time-dependent should be updated accordingly. Similarly, it is assumed that overlay load is simultaneously applied with the barrier load for simplicity.

- Barrier load effect

$$w_{b2} = 0.04 \text{ kips/in.}$$

$$\text{if } x \leq 5.5 \quad M_{b2sw}(x) = -0.02 \cdot x^2 \quad \text{E-229}$$

$$\text{if } 5.5 < x \quad M_{b2sw}(x) = 32.636 \cdot x - 0.02 \cdot (x - 5.5)^2 - 180.103 \quad \text{E-230}$$

$$\Delta\phi_{b2sw}(x) = \frac{M_{b2sw}(x)}{E_{cG}(7) \cdot I_{c2}(7, x)} \quad \text{E-231}$$

- Overlay load effect

$$w_{o2} = t_{o2} \cdot s_d \cdot \gamma_{o2} = \frac{2 \cdot 100.92 \cdot 0.140}{12^3} = 0.0164 \text{ kips/in.}$$

$$\text{if } x \leq 5.5 \quad M_{o2sw}(x) = -8.18 \cdot 10^{-3} \cdot x^2 \quad \text{E-232}$$

$$\text{if } 5.5 < x \quad M_{o2sw}(x) = 13.34 \cdot x - 8.18 \cdot 10^{-3} \cdot (x - 5.5)^2 - 73.43 \quad \text{E-233}$$

$$\Delta\phi_{o2sw}(x) = \frac{M_{o2sw}(x)}{E_{cG}(8) \cdot I_{c2}(8, x)} \quad \text{E-234}$$

Table E.96. Moment applied on 2nd composite structure due to barrier and overlay loads

$M_{b2sw}(x) \text{ (kips} \cdot \text{in.)}$			$M_{o2sw}(x) \text{ (kips} \cdot \text{in.)}$		
$M_{b2sw}(1)$	$M_{b2sw}(2)$	$M_{b2sw}(3)$	$M_{o2sw}(1)$	$M_{o2sw}(2)$	$M_{o2sw}(3)$
0	9940	13313	0	4064	5443

Table E.97. Change in curvature on 2nd super-structure due to barrier and overlay load

$\Delta\phi_{b2sw}(x) \text{ (1/in.)}$			$\Delta\phi_{o2sw}(x) \text{ (1/in.)}$		
$\Delta\phi_{b2sw}(1)$	$\Delta\phi_{b2sw}(2)$	$\Delta\phi_{b2sw}(3)$	$\Delta\phi_{o2sw}(1)$	$\Delta\phi_{o2sw}(2)$	$\Delta\phi_{o2sw}(3)$
0	2.06E-06	2.74E-06	0	8.42E-07	1.12E-06

- Calculation of camber just after application of barrier and overlay loads on 2nd super-structure

By summing up the curvature changes given in Table E.92 and previous net curvature values in Table E.94, the net curvature values just after application of barrier and overlay loads on 2nd super-structure can be obtained as follows:

Table E.98. Net curvature just after application of barrier and overlay loads on 2nd super-structure

$\phi_{naas2}(x) \text{ (1/in.)}$		
$\phi_{naas2}(1)$	$\phi_{naas2}(2)$	$\phi_{naas2}(3)$
3.73E-06	-5.90E-06	3.10E-06

After obtaining the net curvature values along the half-span, camber just after application of barrier and overlay loads on 2nd super-structure can be estimated using Eqns. between E-52 and E-59.

Table E.99. Camber just after application of barrier and overlay loads on 2nd super-structure

Camber (in.)	<i>section index (x)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{naas2}(x)$	0	-0.49	-0.98

E.14. Camber at Final Time

The effect of barrier and overlay loads on girder and deck should be determined and force histories for both deck and girder should be updated before conducting time-dependent analysis. To do so, a similar approach stated in Part 8 is followed. Equations used in section E.8 are valid in this part, however, parameters that are time-dependent should be updated accordingly.

- Time-dependent effect between t_6 and t_7

Table E.100. Change in forces on girder and deck due to time-dependent effects between t_6 and t_7

$\Delta N_{Gt_6t_7}(x) (kips)$			$\Delta M_{Gt_6t_7}(x) (kips - in.)$		
$\Delta N_{Gt_6t_7}(1)$	$\Delta N_{Gt_6t_7}(2)$	$\Delta N_{Gt_6t_7}(3)$	$\Delta M_{Gt_6t_7}(1)$	$\Delta M_{Gt_6t_7}(2)$	$\Delta M_{Gt_6t_7}(3)$
0	-1	-1	-3	-18	-20
$\Delta N_{Dt_6t_7}(x) (kips)$			$\Delta M_{Dt_6t_7}(x) (kips - in.)$		
$\Delta N_{Dt_6t_7}(1)$	$\Delta N_{Dt_6t_7}(2)$	$\Delta N_{Dt_6t_7}(3)$	$\Delta M_{Dt_6t_7}(1)$	$\Delta M_{Dt_6t_7}(2)$	$\Delta M_{Dt_6t_7}(3)$
0	0	0	0	0	0

- Barrier load effect

$$\Delta N_{Gb2sw}(x) = \frac{M_{b2sw}(x) \cdot A_G \cdot [y_{c2}(7, x) - y_{Gbottom}]}{I_{c2}(7, x)} \quad E-235$$

$$\Delta M_{Gb2sw}(x) = \frac{M_{b2sw}(x) \cdot I_G}{I_{c2}(7, x)} \quad E-236$$

$$\Delta N_{d2b2sw}(x) = \frac{M_{b2sw}(x) \cdot A_{d2tr}(7, x) \cdot [y_{c2}(7, x) - y_{d2bottom}]}{I_{c2}(7, x)} \quad E-237$$

$$\Delta M_{d2b2sw}(x) = \frac{M_{b2sw}(x) \cdot I_{d2tr}(7, x)}{I_{c2}(7, x)} \quad E-238$$

Table E.101. Change in forces on girder and deck due to barrier load

$\Delta N_{Gb2sw}(x) (kips)$			$\Delta M_{Gb2sw}(x) (kips - in.)$		
$\Delta N_{Gb2sw}(1)$	$\Delta N_{Gb2sw}(2)$	$\Delta N_{Gb2sw}(3)$	$\Delta M_{Gb2sw}(1)$	$\Delta M_{Gb2sw}(2)$	$\Delta M_{Gb2sw}(3)$
0	123	162	0	4961	6599
$\Delta N_{d2b2sw}(x) (kips)$			$\Delta M_{d2b2sw}(x) (kips - in.)$		
$\Delta N_{d2b2sw}(1)$	$\Delta N_{d2b2sw}(2)$	$\Delta N_{d2b2sw}(3)$	$\Delta M_{d2b2sw}(1)$	$\Delta M_{d2b2sw}(2)$	$\Delta M_{d2b2sw}(3)$
0	-143	-191	0	45	60

- Overlay load effect

$$\Delta N_{Go2sw}(x) = \frac{M_{o2sw}(x) \cdot A_G \cdot [y_{c2}(8, x) - y_{Gbottom}]}{I_{c2}(8, x)} \quad E-239$$

$$\Delta M_{Go2sw}(x) = \frac{M_{o2sw}(x) \cdot I_G}{I_{c2}(8, x)} \quad E-240$$

$$\Delta N_{d2o2sw}(x) = \frac{M_{o2sw}(x) \cdot A_{d2tr}(8, x) \cdot [y_{c2}(8, x) - y_{d2bottom}]}{I_{c2}(4, x)} \quad E-241$$

$$\Delta M_{d2o2sw}(x) = \frac{M_{o2sw}(x) \cdot I_{d2tr}(8, x)}{I_{c2}(8, x)} \quad \text{E-242}$$

Table E.102. Change in forces on girder and deck due to overlay load

$\Delta N_{Go2sw}(x)$ (kips)			$\Delta M_{Go2sw}(x)$ (kips – in.)		
$\Delta N_{Go2sw}(1)$	$\Delta N_{Go2sw}(2)$	$\Delta N_{Go2sw}(3)$	$\Delta M_{Go2sw}(1)$	$\Delta M_{Go2sw}(2)$	$\Delta M_{Go2sw}(3)$
0	50	66	0	2028	2698
$\Delta N_{d2o2sw}(x)$ (kips)			$\Delta M_{d2o2sw}(x)$ (kips – in.)		
$\Delta N_{d2o2sw}(1)$	$\Delta N_{d2o2sw}(2)$	$\Delta N_{d2o2sw}(3)$	$\Delta M_{d2o2sw}(1)$	$\Delta M_{d2o2sw}(2)$	$\Delta M_{d2o2sw}(3)$
0	-59	-78	0	18	24

By summing up the change in forces for deck and girder separately due to abovementioned effects, the force history array for them can be updated as shown in Table E.103.

Table E.103. Net force history array just after application of barrier and overlay loads

<i>index</i>	$N_G(t, x)$ (kips)			$M_G(t, x)$ (kips – in.)		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>1</u>	0	-2677	-2683	0	-32590	-29306
<u>2</u>	57	158	132	820	24011	30343
<u>3</u>	7	145	181	98	5404	6992
<u>4</u>	0	50	66	0	2025	2693
<u>5</u>	55	12	-50	790	-22820	-31594
<u>6</u>	0	-36	-52	-6	18859	25076
<u>7</u>	0	122	162	-3	4943	6578
<u>8</u>	0	50	66	0	2028	2698
<i>index</i>	$N_D(t, x)$ (kips)			$M_D(t, x)$ (kips – in.)		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>1</u>	0	0	0	0	0	0
<u>2</u>	0	0	0	0	0	0
<u>3</u>	0	0	0	0	0	0
<u>4</u>	0	0	0	0	0	0
<u>5</u>	0	0	0	0	0	0
<u>6</u>	0	0	0	0	0	0
<u>7</u>	0	-143	-191	0	45	60
<u>8</u>	0	-59	-78	0	18	13

It should be noted that although there is no time-dependent change in forces between t_7 and t_8 , an additional row is added for this in Table E.103. This is done on purpose so that the time

indices match with the *MDOTCamber* program where the effects of barrier and overlay loads are considered separately. After obtaining the force history array for deck and girder, using Eqns. between E-60 and E-68, and E-104 and E-157, time-dependent calculation can be performed between time t_8 and t_9 . Since it is done explicitly in section E.8, it is not repeated here. After conducting time-dependent calculation, the following change in force and curvature values can be obtained.

Table E.104. Change in forces on girder and deck due to time-dependent effects between t_8 and t_9

$\Delta N_{Gt_8t_9}(x) \text{ (kips)}$			$\Delta M_{Gt_8t_9}(x) \text{ (kips - in.)}$		
$\Delta N_{Gt_8t_9}(1)$	$\Delta N_{Gt_8t_9}(2)$	$\Delta N_{Gt_8t_9}(3)$	$\Delta M_{Gt_8t_9}(1)$	$\Delta M_{Gt_8t_9}(2)$	$\Delta M_{Gt_8t_9}(3)$
-355	-432	-458	10921	12096	12489
$\Delta N_{Dt_8t_9}(x) \text{ (kips)}$			$\Delta M_{Dt_8t_9}(x) \text{ (kips - in.)}$		
$\Delta N_{Dt_8t_9}(1)$	$\Delta N_{Dt_8t_9}(2)$	$\Delta N_{Dt_8t_9}(3)$	$\Delta M_{Dt_8t_9}(1)$	$\Delta M_{Dt_8t_9}(2)$	$\Delta M_{Dt_8t_9}(3)$
350	403	422	49	22	13

Table E.105. Change in curvature between time t_8 and t_9

$\Delta \phi_t(x) \text{ (1/in.)}$		
$\Delta \phi_t(1)$	$\Delta \phi_t(2)$	$\Delta \phi_t(3)$
6.30E-06	8.05E-06	8.53E-06

Using the previous net curvature values given in Table E.98 and change in curvature values provided in Table E.105, the net curvature at final time can be calculated as follows:

Table E.106. Net curvature at final considered time

$\phi_t(9, x) \text{ (1/in.)}$		
$\phi_t(9,1)$	$\phi_t(9,2)$	$\phi_t(9,3)$
1.00E-05	2.15E-06	1.16E-05

Using the net curvature values in Table E.106, and Eqns. between E-52 and E-59, the net camber at considered final time which is 75 years in this example can be estimated as follows:

Table E.107. Camber at final considered time

Camber (in.)	<i>section index (x)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{ns2}(x)$	0	0.90	1.79

E.15. Nomenclature

A_G = girder area, ($in.^2$),

A_{nm} = area under curvature diagram between n^{th} section and midspan, ($in.^2$)

A_n = area under curvature diagram between n^{th} section and closest girder end, ($in.^2$)

A_{psi} = nominal area for a strand, ($in.^2$),

A_{ps} = effective area of strands as a function of location, ($in.^2$),

A_{psdb} = total debonded strand area, ($in.^2$),

A_{pshp} = total harped strand area as a function of location, ($in.^2$),

A_{psst} = total straight strand area, ($in.^2$),

A_{d1} = area of the first deck, ($in.^2$),

A_{nctr} = transformed area of non-composite structure, ($in.^2$),

A_{ctr} = transformed area of composite structure, ($in.^2$),

A_{c1tr} = transformed area of 1st composite structure, ($in.^2$),

A_{c2tr} = transformed area of 2nd composite structure, ($in.^2$),

A = net area of the element that is going to be transformed, ($in.^2$),

A_d = area of the deck, ($in.^2$),

A_{pstr} = transformed area of the prestressing strand, ($in.^2$),

A_{dtr} = transformed area for the deck, ($in.^2$),

A_{d1tr} = transformed area for the 1st deck, ($in.^2$),

A_{d2tr} = transformed area for the 2nd deck, ($in.^2$),

b_f = girder top flange width, (in.),

Co_{NG} = coefficient related to axial force on the girder,

Co_{MG} = coefficient related to moment on the girder,

Co_{NG} = coefficient related to axial force on the deck,

Co_{MG} = coefficient related to moment on the deck,

d_{ps} = nominal strand diameter, (in.),

e_{db} = eccentricity of debonded strands, ($in.^2$),

$e_{hp,end}$ = eccentricity of harped strands at girder end, (in.),

$e_{hp,mid}$ = eccentricity of harped strands at girder midspan, (in.),

e_{hp} = eccentricity of harped strand as a function of location, (in.),

e_{st} = eccentricity of straight strands, (in.),

e_e = net end eccentricity, (in.),

e_{atdb} = net eccentricity at the end of debonding length, (in.),

e_m = net midspan eccentricity, (in.),

e_{ts} = eccentricity of prestressing strand in transformed non-composite structure, (in.),

e_c = eccentricity of prestressing strand in transformed composite structure, (in.),

e_{c1} = eccentricity of prestressing strand in transformed composite structure with 1st deck, (in.),

e_{c2} = eccentricity of prestressing strand in transformed composite structure with 2nd deck, (in.),

E_c = modulus of elasticity as a function of time, (ksi),

E_{CG} = modulus of elasticity function for girder as a function of time, (ksi),

E_{CD} = the modulus of elasticity function for deck as a function of time, (ksi),

E_{CD1} = the modulus of elasticity function for the first deck as a function of time, (ksi),

E_{CD2} = the modulus of elasticity function for the second deck as a function of time, (ksi),

E'_{c28} = 28-day modulus of elasticity of concrete, (ksi),

E'_{ci} = modulus of elasticity value when strands are released, (ksi),

E_G = modulus of elasticity of girder concrete at a specific time, (ksi),

E_D = modulus of elasticity of deck concrete at a specific time, (ksi),

E_{tr} = modulus of elasticity of the transformed material, (ksi),

f_{py} = yield stress of the prestressing strands, (ksi),

f_{psj} = stress on the strands along the span prior to transfer, (ksi),
 f_{pR} = remaining relaxation loss array as a function of time and location, (kips),
 f'_{cdi} = initial concrete compressive strength for deck, (ksi),
 f'_{cd} = 28-day concrete compressive strength for deck, (ksi),
 f'_{ci} and f'_{ci} are initial concrete compressive strength, (ksi),
 f'_c = 28-day concrete compressive strength, (ksi),
 f_{pj} = jacking stress of the prestressing strands, (ksi),
 g = a time index for the initiation of composite action,
 h = section index array,
 H_r = the average annual ambient relative humidity (%),
 i = index for initial reference time,
 I_{ts} = moment of inertia of the transformed non-composite structure, (in.⁴),
 I_{dtr} = transformed moment of inertia of the deck, (in.⁴),
 I_{d1tr} = transformed moment of inertia of the 1st deck, (in.⁴),
 I_{d2tr} = transformed moment of inertia of the 2nd deck, (in.⁴),
 I_c = transformed moment of inertia of composite structure, (in.⁴),
 I_{c1} = transformed moment of inertia of composite structure with 1st deck, (in.⁴),
 I_{c2} = transformed moment of inertia of composite structure with 2nd deck, (in.⁴),
 j = section index array,
 k = time index array,
 k_s = factor for the effect of the volume-to-surface ratio of the component
 k_{hc} = humidity factor for creep,
 k_f = factor for the effect of concrete strength,
 k_{tdc} = time development factor for creep,
 k_{tds} = time development factor for shrinkage,
 k_{hs} = humidity factor for shrinkage,
 K_L = factor accounting for type of prestressing strand, equal to 45 for low

relaxation strands (AASHTO LRFD 8th Edition section 5.9.3.4.2c)
 K_i = accounts for time-dependent interaction between concrete and bonded steel in the section being considered (AASHTO LRFD 8th Edition section 5.9.3.4.2a-2),
 K_1 = coefficient depending on aggregate type: for normal weight concrete $K_1 = 1.0$,
 L_{tr} = transfer length (in.),
 L_{dbavg} = average debonding length, (in.),
 L_{hpavg} = average harping length, (in.),
 L_{hpi} = harping length for i^{th} harped strand group, (in.),
 L_{dbi} = debonding length for i^{th} debonded strand group, (in.),
 L_t = girder overhang length at precast facility, (in.),
 m = number of sections to be investigated,
 M_{ps} = moment on the non-composite structure due to prestressing effect, (kips – in.),
 M_{Gi} = net moment on the non-composite structure due to prestressing and girder self-weight effect, (kips – in.),
 M_{Gsw} = moment on the non-composite structure due to girder self-weight at precast facility, (kips – in.),
 M_{Gsi} = moment on girder alone due to girder self-weight at precast facility, (kips – in.),
 M_{psst} = moment on the non-composite structure due to straight prestressing strand, (kips – in.),
 M_{psdb} = moment on the non-composite structure due to debonded prestressing strand, (kips – in.),
 M_{pshp} = moment on the non-composite structure due to harped prestressing strand, (kips – in.),
 M_{Gps} = moment on girder only caused by prestressing effect as a function of location, (kips – in.),
 M_{Gp} = net moment on girder only due to combined effect at release, (kips – in.),
 M_{co} = matrix composed of coefficients for time-dependent calculations,

M_c = matrix of constants for time-dependent calculations,

M_s = solution matrix for time-dependent calculations,

M_G = array of moment on the girder a function of time and location, (*kips – in.*),

N_D = array of axial force on the deck a function of time and location, (*kips*),

M_D = array of moment on the deck a function of time and location, (*kips – in.*),

M_{d1sw} = moment caused by 1st deck load as a function of location, (*kips – in.*),

M_{d2sw} = moment caused by 2nd deck load as a function of location, (*kips – in.*),

M_{b1sw} = moment caused by 1st barrier load as a function of location, (*kips – in.*),

M_{b2sw} = moment caused by 2nd barrier load as a function of location, (*kips – in.*),

M_{o1sw} = moment caused by 1st overlay load as a function of location, (*kips – in.*),

M_{o2sw} = moment caused by 2nd overlay load as a function of location, (*kips – in.*),

M_{Gsp} = moment due to girder self-weight on non-composite structure on bridge site, (*kips – in.*),

M_{d1br} = net moment on the 1st deck before it is removed, (*kips – in.*),

M_{pR} = array of moment on non-composite structure caused by remaining relaxation loss as a function of time and location, (*kips – in.*),

N_{st} = total number of straight strand, (*in.*²),

N_{db} = total number of debonded strand, (*in.*²),

N_{hp} = total area of harped strand, (*in.*²),

N_i = number of strands debonded in ith strand group,

N_{ps} = axial force on the non-composite structure due to prestressing effect, (*kips*),

N_{Gi} = net axial force on the non-composite structure due to prestressing and girder self-weight effect, (*kips*),

N_{Gsi} = axial force on girder alone due to girder self-weight at precast facility, (*kips*),

N_{psst} = axial force on the non-composite structure due to straight prestressing strand, (*kips*),

N_{psdb} = axial force on the non-composite structure due to debonded prestressing strand, (*kips*),

N_{pshp} = axial force on the non-composite structure due to harped prestressing strand, (*kips*),

N_{Gps} = axial force on girder only caused by prestressing effect as a function of location, (*kips*),

N_{Gp} = net axial force on girder only due to combined effect at release, (*kips*),

N_G = array of axial force on the girder a function of time and location, (*kips*),

N_{d1br} = net axial force on the 1st deck before it is removed, (*kips*),

N_{tloss} = total time-dependent loss with respect to time along the half-span, (*kips*),

N_{tloss_0} = initial time-dependent loss with respect to time along the half-span, (*kips*),

P_{psst} = axial force on the straight prestressing strand, (*kips*),

P_{psdb} = axial force on the debonded prestressing strand, (*kips*),

P_{pshp} = axial force on the harped prestressing strand, (*kips*),

p = index for final reference time,

Q_n = first moment of the area under curvature diagram between nth section and closest girder end, (*in.*³),

r = helper symbol that is related to time reference for time-dependent calculations,

s_d = average center-to-center spacing of the girders, (*in.*),

S_{d1} = perimeter of the first deck, (*in.*),

t_1 = time when strands are released, (*days*),

t_i = age of concrete at the time of load application (*days*),

t = maturity of concrete, defined as age of concrete between time of loading for creep calculations, or end of curing for shrinkage calculations, and time being considered for

analysis of creep or shrinkage effects, (days),

t_d = average deck thickness, (in.),

t_h = average haunch thickness, (in.),

t_{o1} = 1st overlay thickness, (in.),

t_{o2} = 2nd overlay thickness, (in.),

u = time index array,

V_{d1} = volume of the first deck, (in.³),

V/S = volume-to-surface ratio (in.),

w = unit weight of concrete, (kcf),

w_G = load due to girder self-weight, kips/in.,

w_{d1} = load due to 1st deck, (kips/in.),

w_{d2} = load due to 2nd deck, (kips/in.),

w_{b1} = load due to 1st barrier load, (kips/in.),

w_{b2} = load due to 2nd barrier load, (kips/in.),

w_{o1} = load due to 1st overlay load, (kips/in.),

w_{o2} = load due to 2nd overlay load, (kips/in.),

x_n = location of nth section measured from closest girder end, (in.),

X_n = curvature at nth section, (1/in.),

x_m = location of midspan measured from girder end, (in.),

X_m = curvature at midspan, (1/in.),

$\bar{x}_n$ = centroid of the area under curvature diagram between nth section and closest girder end, (in.),

y_{dG} = a parameter indicating the distance between centroids of girder and deck, (in.),

$y_{hpi,end}$ = distance from centroid of ith harped strand group to the bottom of the girder at end, (in.),

$y_{hpi,mid}$ = distance from centroid of ith harped strand group to the bottom of the girder at midspan, (in.),

$y_{Gbottom}$ = centroid of prestressed concrete beam with respect to bottom of girder, (in.),

y_{dbi} = distance from centroid of ith debonded strand group to the bottom of the girder, (in.),

y_{sti} = distance from centroid of ith straight strand group to the bottom of the girder, (in.),

$y_{psbottom}$ = distance of centroid of prestressing strands to the bottom of girder, (in.),

y_t = distance of centroid of transformed non-composite structure to the bottom of girder, (in.),

y_{dc} = centroid of the deck & haunch couple with respect to the bottom of the haunch, (in.),

$y_{dbottom}$ = distance between centroid of deck & haunch couple with respect to bottom of the girder, (in.),

y_c = distance of centroid of transformed composite structure to the bottom of girder, (in.),

y_{c1} = distance of centroid of transformed composite structure to the bottom of girder with 1st deck, (in.),

y_{c2} = distance of centroid of transformed composite structure to the bottom of girder with 2nd deck, (in.),

z = time index array,

z_d = row number of the axial force history array,

loc = array for location, (in.),

$stopper$ = helper word for time-dependent calculations,

$continue$ = loop term that restarts the current iteration,

α = parameters for ACI209R-92 development model depending on curing type,

β = parameters for ACI209R-92 development model depending on cement type,

$\theta_{B/C}$ = relative slope at point B relative to point C, (rad.),

γ_G = unit-weight of girder including reinforcement weight, (kcf),

γ_{o1} = unit-weight of 1st overlay, (pcf),

γ_{o2} = unit-weight of 2nd overlay, (pcf),

ε_{shG} = remaining girder shrinkage at a given time, (in./in.),

ε_{shD} = remaining deck shrinkage at a given time, (in./in.),

β_{ax} = magnification factor tension creep,

β_{flex} = magnification factor flexural creep,
 ψ_{crG} = remaining girder creep at a given time,
 $\psi_{crG,flex}$ = remaining girder flexural creep at a given time,
 ψ_{crD} = remaining deck creep at a given time,
 $\psi_{crD,flex}$ = remaining deck flexural creep at a given time,
 $\psi_{crG,ax}$ = remaining girder tension creep at a given time,
 $\psi_{crD,ax}$ = remaining deck tension creep at a given time,
 $\psi(t, t_i)$ = the creep coefficient at concrete at t due to load applied at t_i ,
 ϕ_0 = initial curvature array as a function of time and location, (1/in.),
 ϕ_p = initial curvature array as a function of time and location, (1/in.),
 $\Delta\phi_t$ = change in curvature due to time-dependent effects as a function of time and location, (1/in.),
 ϕ_{ad1p} = net curvature just after 1st deck is placed, (1/in.),
 ϕ_{ad2p} = net curvature just after 2nd deck is placed, (1/in.),
 ϕ_{naas1} = net curvature just after application of barrier and overlay loads on 1st superstructure, (1/in.),
 ϕ_{naas2} = net curvature just after application of barrier and overlay loads on 2nd superstructure, (1/in.),
 ϕ_{ad1r} = net curvature just after deck removal, (1/in.),
 η = modular ratio,
 η_{ps} = modular ratio for prestressing strands,
 η_D = modular ratio for the deck
 η_{D1} = modular ratio for the 1st deck,
 η_{D2} = modular ratio for the 2nd deck,
 Δ_{ni} = camber at release (in.),
 Δ_{np} = pre-erection camber (in.),
 Δ_{nad1p} = camber just after placement of 1st deck, (in.),
 Δ_{nbas1} = camber just before application of barrier and overlay loads on 1st superstructure, (in.),

Δ_{nbas2} = camber just before application of barrier and overlay loads on 2nd superstructure, (in.),
 Δ_{naas1} = camber just after application of barrier and overlay loads on 1st superstructure, (in.),
 Δ_{naas2} = camber just after application of barrier and overlay loads on 2nd superstructure, (in.),
 Δ_{nbd1r} = camber just before deck removal process, (in.),
 Δ_{nad1r} = camber just after deck removal process, (in.),
 Δ_{nbndp} = camber just before new deck placement, (in.),
 Δ_{nad2p} = camber just after placement of 2nd deck, (in.),
 Δ_{ns2} = camber at final considered time, (in.),
 Δ_n = net deflection at nth section according to end of the girder, (in.),
 $\Delta_{A/C}$ = relative displacement at point C according to point A, (in.),
 Δx = length of a section, (in.),
 δN_{GpR} = axial effect of relaxation loss on girder as a function of time and location, (kips),
 δN_{GpR} = axial effect of relaxation loss on deck as a function of time and location, (kips),
 δM_{GpR} = moment effect of relaxation loss on girder as a function of time and location, (kips – in.),
 δM_{DpR} = moment effect of relaxation loss on deck as a function of time and location, (kips – in.),
 Δf_{pR} = relaxation loss array as a function of time and location, (kips),
 Δf_{pR0} = initial relaxation loss array as a function of time and location, (kips),
 Δf_{pES} = elastic shortening loss array as a function of time and location, (ksi),
 ΔM_{Gob1r} = change in moment on the girder due to removal of barrier and overlay loads, (kips – in.),

ΔM_{d1ob1r} = change in moment on the 1st deck due to removal of barrier and overlay loads, (*kips – in.*),

ΔM_{Gd1swr} = change in moment on girder due to deck self-weight removal, (*kips – in.*),

ΔM_{tsd1r} = change in moment on non-composite structure due to removal of locked forces inside the 1st deck, (*kips – in.*),

ΔM_{Gtsd1r} = change in moment on girder due to removal of locked forces inside the 1st deck, (*kips – in.*),

ΔM_{Gsc} = change in moment on the girder due to change in support condition, (*kips – in.*),

ΔM_{Gd1sw} = change in moment on the girder due to 1st deck load, (*kips – in.*),

ΔM_{Gd2sw} = change in moment on the girder due to 2nd deck load, (*kips – in.*),

ΔM_{Gb1sw} = change in moment on the girder due to 1st barrier load, (*kips – in.*),

ΔM_{d1b1sw} = change in moment on the 1st deck due to 1st barrier load, (*kips – in.*),

ΔM_{Gb2sw} = change in moment on the girder due to 2nd barrier load, (*kips – in.*),

ΔM_{d2b2sw} = change in moment on the 2nd deck due to 2nd barrier load, (*kips – in.*),

ΔM_{Go1sw} = change in moment on the girder due to 1st overlay load, (*kips – in.*),

ΔM_{d1o1sw} = change in moment on the 1st deck due to 1st overlay load, (*kips – in.*),

ΔM_{Go2sw} = change in moment on the girder due to 2nd overlay load, (*kips – in.*),

ΔM_{d2o2sw} = change in moment on the 2nd deck due to 2nd overlay load, (*kips – in.*),

$\Delta M_{Gt_1t_2}$ = change in moment on girder due to time-dependent effects between t_1 and t_2 , (*kips – in.*),

$\Delta M_{Gt_2t_3}$ = change in moment on girder due to time-dependent effects between t_2 and t_3 , (*kips – in.*),

$\Delta M_{Dt_2t_3}$ = change in moment on deck due to time-dependent effects between t_2 and t_3 , (*kips – in.*),

$\Delta M_{Gt_4t_5}$ = change in moment on girder due to time-dependent effects between t_4 and t_5 , (*kips – in.*),

$\Delta M_{Dt_4t_5}$ = change in moment on deck due to time-dependent effects between t_4 and t_5 , (*kips – in.*),

$\Delta M_{Gt_5t_6}$ = change in moment on girder due to time-dependent effects between t_5 and t_6 , (*kips – in.*),

$\Delta M_{Gt_6t_7}$ = change in moment on girder due to time-dependent effects between t_6 and t_7 , (*kips – in.*),

$\Delta M_{Dt_6t_7}$ = change in moment on deck due to time-dependent effects between t_6 and t_7 , (*kips – in.*),

$\Delta M_{Gt_8t_9}$ = change in moment on girder due to time-dependent effects between t_8 and t_9 , (*kips – in.*),

$\Delta M_{Dt_8t_9}$ = change in moment on deck due to time-dependent effects between t_8 and t_9 , (*kips – in.*),

ΔN_{Gob1r} = change in axial force on the girder due to removal of barrier and overlay loads, (*kips*),

ΔN_{d1ob1r} = change in axial force on the 1st deck due to removal of barrier and overlay loads, (*kips*),

ΔN_{Gd1swr} = change in axial force on girder due to deck self-weight removal, (*kips*),

ΔN_{tsd1r} = change in axial force on non-composite structure due to removal of locked forces inside the 1st deck, (*kips*),

ΔN_{Gtsd1r} = change in axial force on girder due to removal of locked forces inside the 1st deck, (*kips*),

ΔN_{Gsc} = change in axial force on the girder due to change in support condition, (*kips*),

ΔN_{Gd1sw} = change in axial force on the girder due to 1st deck load, (*kips*),

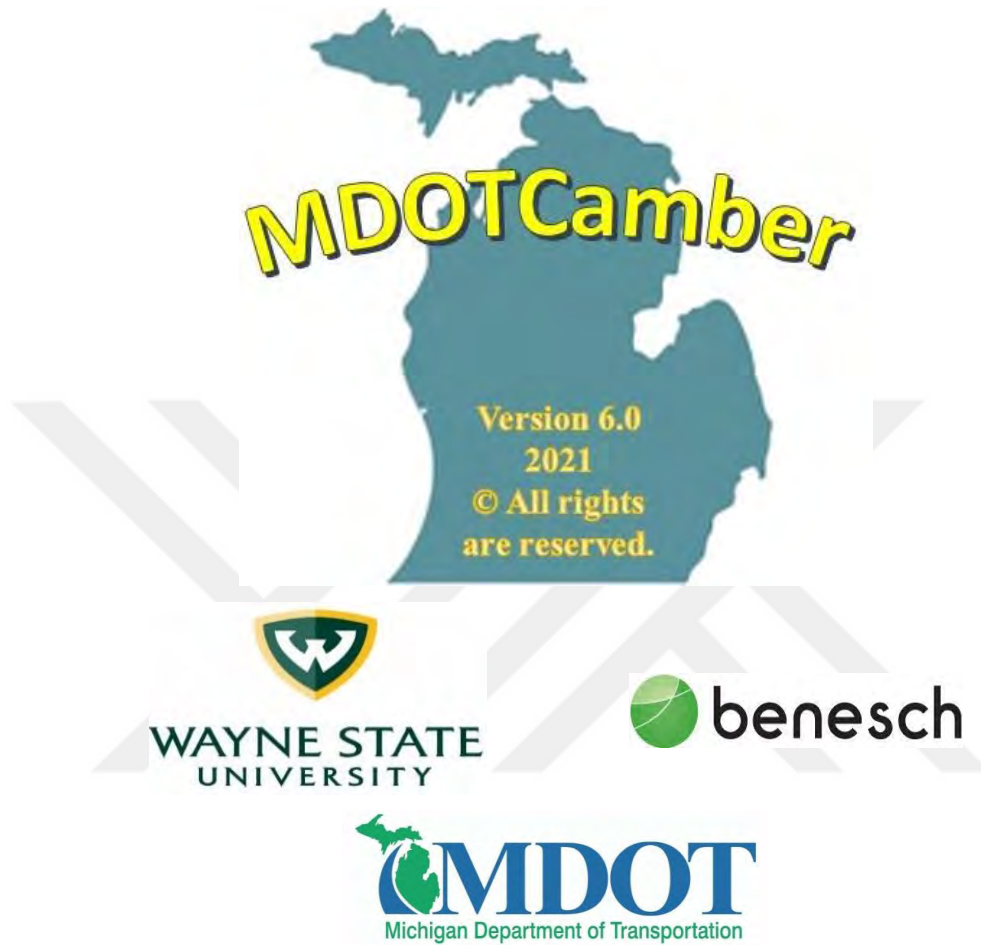
ΔN_{Gd2sw} = change in axial force on the girder due to 2nd deck load, (*kips*),

ΔN_{Gb1sw} = change in axial force on the girder due to 1st barrier load, (*kips*),

ΔN_{d1b1sw} = change in axial force on the 1st deck due to 1st barrier load, (*kips*),

ΔN_{Gb2sw} = change in axial force on the girder due to 2nd barrier load, (*kips*),
 ΔN_{d2b2sw} = change in axial force on the 2nd deck due to 2nd barrier load, (*kips*),
 ΔN_{Go1sw} = change in axial force on the girder due to 1st overlay load, (*kips*),
 ΔN_{d1o1sw} = change in axial force on the 1st deck due to 1st overlay load, (*kips*),
 ΔN_{Go2sw} = change in axial force on the girder due to 2nd overlay load, (*kips*),
 ΔN_{d2o2sw} = change in axial force on the 2nd deck due to 2nd overlay load, (*kips*),
 $\Delta N_{Gt_1t_2}$ = change in axial force on girder due to time-dependent effects between t_1 and t_2 , (*kips*),
 $\Delta N_{Gt_2t_3}$ = change in axial force on girder due to time-dependent effects between t_2 and t_3 , (*kips*),
 $\Delta N_{Dt_2t_3}$ = change in axial force on deck due to time-dependent effects between t_2 and t_3 , (*kips*),
 $\Delta N_{Gt_4t_5}$ = change in axial force on girder due to time-dependent effects between t_4 and t_5 , (*kips*),
 $\Delta N_{Dt_4t_5}$ = change in axial force on deck due to time-dependent effects between t_4 and t_5 , (*kips*),
 $\Delta N_{Gt_5t_6}$ = change in axial force on girder due to time-dependent effects between t_5 and t_6 , (*kips*),
 $\Delta N_{Gt_6t_7}$ = change in axial force on girder due to time-dependent effects between t_6 and t_7 , (*kips*),
 $\Delta N_{Dt_6t_7}$ = change in axial force on deck due to time-dependent effects between t_6 and t_7 , (*kips*),
 $\Delta N_{Gt_8t_9}$ = change in axial force on girder due to time-dependent effects between t_8 and t_9 , (*kips*),
 $\Delta N_{Dt_8t_9}$ = change in axial force on deck due to time-dependent effects between t_8 and t_9 , (*kips*),
 $\Delta \phi_{sc}$ = change in curvature due to change in support conditions, (*1/in.*),

$\Delta \phi_{d1sw}$ = change in curvature due to 1st deck self-weight, (*1/in.*),
 $\Delta \phi_{d2sw}$ = change in curvature due to 2nd deck self-weight, (*1/in.*),
 $\Delta \phi_{b1sw}$ = change in curvature due to 1st barrier self-weight, (*1/in.*),
 $\Delta \phi_{b2sw}$ = change in curvature due to 2nd barrier self-weight, (*1/in.*),
 $\Delta \phi_{o1sw}$ = change in curvature due to 1st barrier self-weight, (*1/in.*),
 $\Delta \phi_{o2sw}$ = change in curvature due to 2nd barrier self-weight, (*1/in.*),
 $\Delta \phi_{ob1r}$ = change in curvature due to removal of barrier and overlay loads, (*1/in.*),
 $\Delta \phi_{d1r}$ = change in curvature due to removal of locked forces inside the deck, (*1/in.*),
 $\Delta \phi_{d1swr}$ = change in curvature due to deck self-weight removal, (*1/in.*)

APPENDIX F: Example Problem for Proposed Prediction Methodology for Steel Girders**Research Team****Wayne State University:**

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F.1. Definition of the Problem

Using the proposed prediction methodology, Approach No. 6, explained in this thesis, camber and displacement along the span is going to be estimated at:

1. time at which the deck is placed (t_1);
2. time at placement of barriers and the opening of bridge to traffic (t_2);
3. time at placement of overlay (t_3);
4. time at deck removal for projects that features a deck replacement (t_4);
5. time at new deck placement (t_5);
6. time at reopening of bridge to traffic (t_6);
7. time at placement of overlay (t_7);
8. time at the end of service life (t_8);

for the following steel beam used in M20 project:

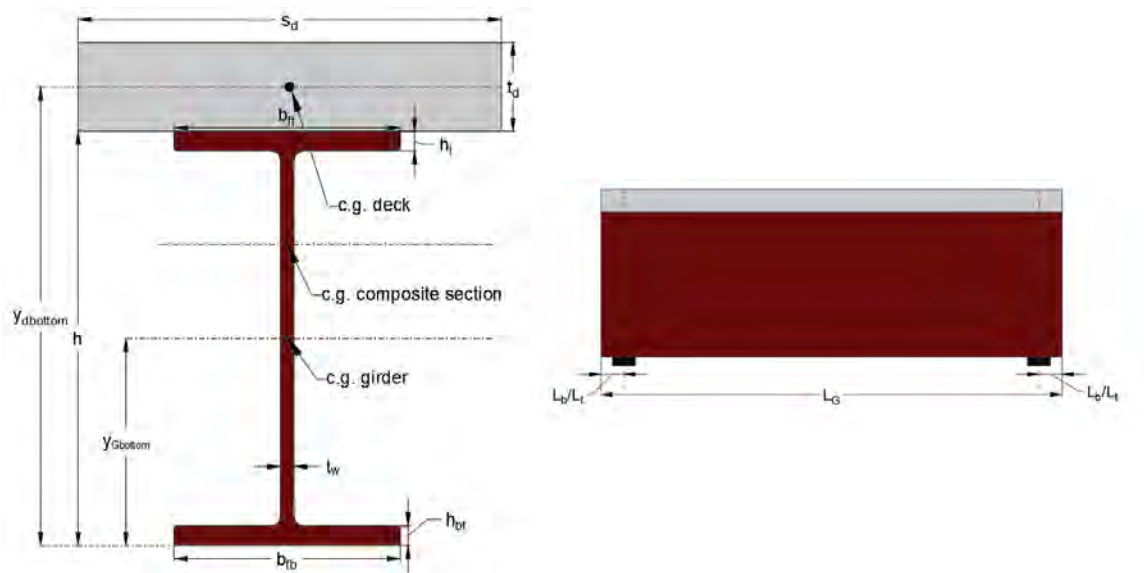


Figure F.1. Notation used for steel girder

Parameters related to girder and project specific properties are provided in Table F.1.

Table F.1. Girder and project specific properties

Parameter	Value
Gross Area (A_G) ($in.^2$)	90
Unit-weight of cross-section (γ_G) (kcf)	0.490
Girder length (L_G) ($ft.$)	173
Centroid of girder measured from bottom ($y_{Gbottom}$) ($in.$)	35.5
Gross moment of inertia (I_G) ($in.^4$)	75840
Modulus of elasticity (E_G) (ksi)	28500
Girder overhang length at bridge site (L_b) ($in.$)	24
Girder height (h) ($in.$)	72
Top flange width (b_{ft}) ($in.$)	23
Top flange thickness (h_t) ($in.$)	1
Web thickness (t_w) ($in.$)	0.625
Bottom flange width (b_{fb}) ($in.$)	23
Bottom flange thickness (h_{bt}) ($in.$)	1
Girder tributary width* (s_d) ($ft.$)	9.17
Thermal expansion coefficient (α) ($1/^\circ F$)	6.7E-06
Fabricated camber** (Δ_{G0}) ($in.$)	-14.64
*For the fascia beams, spacing should be adjusted accordingly.	
**Fabricated camber does not include the deflection caused by girder self-weight.	

Moreover, parameters related of 1st and 2nd super-structures are provided in Table F.2.

Table F.2. Parameters related to first and second decks

Parameter for the First Deck	Value
Specified concrete compressive strength (f'_{cd1}) (ksi)	4
Unit-weight of deck and haunch* (γ_{d1}) (pcf)	0.150
Average deck thickness** (t_{d1}) (in.)	9
Average haunch thickness (t_{h1}) (in.)	0
Overlay thickness (t_{o1}) (in.)	2
Unit-weight of overlay (γ_{o1}) (pcf)	0.140
Barrier load (w_{b1}) (klf)	0.475
Aggregate factor [†] (K_1)	1
Unit-weight of plain concrete [†] (w_{c1}) (pcf)	0.145
Parameter for the Second Deck	Value
Specified concrete compressive strength (f'_{cd2}) (ksi)	4
Unit-weight of deck and haunch* (γ_{d2}) (pcf)	0.150
Average deck thickness** (t_{d2}) (in.)	9
Average haunch thickness (t_{h2}) (in.)	0
Overlay thickness (t_{o2}) (in.)	2
Unit-weight of overlay (γ_{o2}) (pcf)	0.140
Barrier load (w_{b2}) (klf)	0.475
Aggregate factor [†] (K_1)	1
Unit-weight of plain concrete [†] (w_{c2}) (pcf)	0.145
*Reinforcement weight is considered as 5 pcf.	
**For fascia beam analysis, deck thickness should be adjusted accordingly.	
[†] These are used for AAHSTO LRFD (2020) 9 th Edition elastic modulus calculations.	

In this example, the following time arrangement is going to be utilized. It should be noted that the barrier and overlay load is applied to super-structure simultaneously so that there is no additional time-dependent activity taking place between these two events.

Table F.3. Time arrangement for major activities

Event	<i>time index (t)</i>	<i>t(t) (days)</i>
Time at which the deck is placed	<u>1</u>	2
Time at placement of barriers and the opening of bridge to traffic	<u>2</u>	9
Time at placement of overlay	<u>3</u>	9
Time at deck removal for projects that features a deck replacement	<u>4</u>	18250
Time at new deck placement	<u>5</u>	18260
Time at reopening of bridge to traffic	<u>6</u>	18267
Time at placement of overlay	<u>7</u>	18267
Time at the end of service life	<u>8</u>	27375

For the analysis, it is assumed that both decks (first and second) subject to 7 days of moist curing, and Type I cement is used in concrete composition. In addition, it is assumed that deck concrete reaches 80% of its 28-day strength at the end of curing period. ACI 209R-92 model is used with corresponding parameters for the development of modulus of elasticity with time. In addition, AASHTO LRFD (2020) equation is utilized for the estimation of modulus of elasticity. For conducting time-dependent analysis, AASHTO LRFD (2020) Body creep and shrinkage models are employed by assuming the flexural and tensile creep are equal to compression creep. In addition to this, three sections are considered along the half-span to minimize the computation effort. To estimate the deflection from curvature diagram, Simpson's rule is employed for numerical integration. It is assumed that the ambient relative humidity is 70% for Michigan based on the map provided in AASHTO LRFD (2020) 9th Edition Figure 5.4.2.3.3-1. While generating the time matrix for the analysis, only one time-step is going to be taken between the major activities as stated above. Finally, the sign convention adopted in this example problem is given in the Figure F.2.

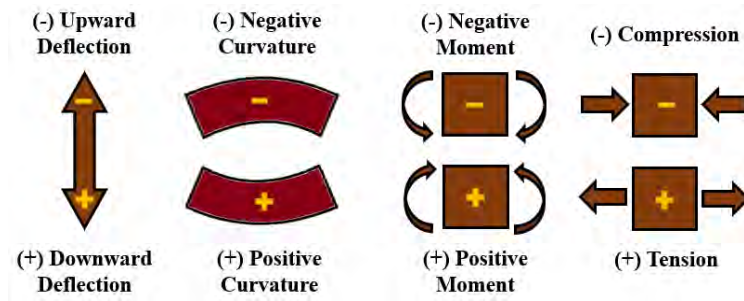


Figure F.2. Sign convention

F.2. Definition of Parameters

- *Generate the time and section matrix*

Since only one time step is going to be considered in this example, the time matrix generated for this specific example would be same as in Table F.3. In addition to this, since three sections are going to be considered along the half-span, the section matrix can be assigned as follows (Table F.4):

Table F.4. Section array

<i>section index (x)</i>	<i>loc(x) (in.)</i>
<u>1</u>	0
<u>2</u>	519
<u>3</u>	1038

It should be noted that when creating time and section dependent arrays, index number is going to be used for simplicity throughout the analysis. For example, say a parameter $A(t, x)$ is both time and section-dependent. $A(6,3)$ refers to the value of A at time $t(t) = t(6) = 18267$ days and at location $loc(x) = loc(3) = 1038$ in. from girder end.

- *Generate creep and shrinkage arrays*
- Creep function based on AASHTO LRFD (2020) 9th edition

$$\psi(t, t_i) = 1.9 \cdot k_s \cdot k_{hc} \cdot k_f \cdot k_{td} \cdot t_i^{-0.118} \quad \text{F-1}$$

$$k_s = 1.45 - 0.13 \cdot \left(\frac{V}{S}\right) \geq 1.0 \quad \text{F-2}$$

$$k_{hc} = 1.56 - 0.008 \cdot H_r \quad \text{F-3}$$

$$k_f = \frac{5}{1 + f'_{cdi}} \quad \text{F-4}$$

$$k_{td}(t) = \frac{t}{12 \cdot \left(\frac{100 - 4 \cdot f'_{cdi}}{f'_{cdi} + 20} \right) + t} \quad \text{F-5}$$

- Shrinkage function based on AASHTO LRFD (2020) 9th edition

$$\varepsilon_{sh}(t) = k_s \cdot k_{hs} \cdot k_f \cdot k_{td} \cdot 0.00048 \quad \text{F-6}$$

$$k_{hs} = 2.00 - 0.014 \cdot H_r \quad \text{F-7}$$

Here, the volume-to-surface ratio can also be expressed as surface-to-perimeter ratio. Moreover, according to AASHTO LRFD (2020) Bridge Design Specification section 5.4.2.3.3, shrinkage as determined in Eqn. F-6 should be increased by 20 % if the concrete is exposed to drying before 5 days of curing have elapsed.

- *Creep and shrinkage calculation for 1st deck*

Since initial concrete compressive strength for the deck is generally unknown, it is assumed:

$$f'_{cdi} = 0.80 \cdot f'_{cd} \quad \text{F-8}$$

$$f'_{cdi} = 0.80 \cdot 4 = 3.2 \text{ ksi}$$

It should be noted that since the deck is 7-days moist-cured, there will be no shrinkage that takes places within the 7-days period after it is cast.

$$A_{d1} = s_d \cdot t_{d1} + t_{h1} \cdot b_f \quad \text{F-9}$$

$$S_{d1} = 2 \cdot s_d - b_f + t_{h1} \quad \text{F-10}$$

$$\frac{V_{d1}}{A_{d1}} = \frac{A_{d1}}{S_{d1}} \quad \text{F-11}$$

$$A_{d1} = (9.17 \cdot 12) \cdot 9 = 990.4 \text{ in.}^2$$

$$S_{d1} = 2 \cdot (9.17 \cdot 12) - 23 + 0 = 197.1 \text{ in.}$$

$$k_s = 1.45 - 0.13 \cdot \left(\frac{990.4}{197.1} \right) = 0.797 < 1.0 \rightarrow k_s = 1.0$$

$$k_{hc} = 1.56 - 0.008 \cdot (70) = 1.00$$

$$k_{hs} = 2.00 - 0.014 \cdot (70) = 1.02$$

$$k_f = \frac{5}{1 + 3.2} = 1.190$$

$$k_{tdc}(t, t_i) = \frac{t - t_i}{12 \cdot \left(\frac{100 - 4 \cdot (3.2)}{3.2 + 20} \right) + (t - t_i)} = \frac{t - t_i}{45.1 + (t - t_i)}$$

$$k_{tds}(t, t_i) = \frac{t - 2 - 7}{12 \cdot \left(\frac{100 - 4 \cdot (3.2)}{3.2 + 20} \right) + (t - 2 - 7)} = \frac{t - 9}{36.1 + t}$$

Using Eqns. F-1 and F-6, the creep and shrinkage array for the 1st deck can be obtained as provided in Table F.5 and Table F.6. To exemplify, $\psi_{D1}(t_8, t_3) = \psi_{D1}(27375, 9)$ and $\varepsilon_{shD1}(t_8) = \varepsilon_{shD1}(27375)$ can be calculated as shown below.

$$\psi_{D1}(27375, 9) = 1.9 \cdot 1.0 \cdot 1.0 \cdot 1.19 \cdot \frac{27375 - 9}{45.1 + (27375 - 9)} \cdot (9 - 2)^{-0.118} = 1.79$$

$$\varepsilon_{shD1}(27375) = 1.0 \cdot 1.02 \cdot 1.19 \cdot \frac{27375 - 9}{36.1 + 27375} \cdot 0.00048 = 581.67 \mu\varepsilon$$

Table F.5. Creep function for 1st deck

$\psi_{D1}(t, t_i)$		t (days)					
		<u>2</u>	<u>9</u>	<u>18250</u>	<u>18260</u>	<u>18627</u>	<u>27375</u>
t_i (days)	<u>2</u>	0.00	0.00	0.00	0.00	0.00	0.00
	<u>9</u>	-	0.00	1.79	1.79	1.79	1.79
	<u>18250</u>	-	-	0.00	0.13	0.19	0.71
	<u>18260</u>	-	-	-	0.00	0.01	0.71
	<u>18627</u>	-	-	-	-	0.00	0.71
	<u>27375</u>	-	-	-	-	-	0.00
t_i = loading time, t = desired time							

Table F.6. Shrinkage array for 1st deck

Event	<i>time index</i> (t)	$\epsilon_{shD1}(t)$ ($\mu\epsilon$)
Time at which the deck is placed	<u>1</u>	0
Time at placement of barriers and the opening of bridge to traffic	<u>2</u>	0
Time at placement of overlay	<u>3</u>	0
Time at deck removal for projects that features a deck replacement	<u>4</u>	581
Time at new deck placement	<u>5</u>	581
Time at reopening of bridge to traffic	<u>6</u>	581
Time at placement of overlay	<u>7</u>	581
Time at the end of service life	<u>8</u>	581

▪ *Creep and shrinkage calculation for 2nd deck*

Similar to the 1st deck, the creep and shrinkage arrays for the 2nd deck can also be determined. For simplicity, it is assumed that the 2nd deck has exactly the same features with 1st deck. The only formulation that is different for the 2nd deck is as follows:

$$k_{tds}(t) = \frac{t - 18260 - 7}{12 \cdot \left(\frac{100 - 4 \cdot (3.2)}{3.2 + 20} \right) + (t - 18260 - 7)} = \frac{t - 18267}{t - 18221.9}$$

Table F.7. Creep function for 2nd deck

$\psi_{D2}(t, t_i)$		t (days)		
		<u>18260</u>	<u>18627</u>	<u>27375</u>
t_i (days)	<u>18260</u>	0.00	0.00	0.00
	<u>18627</u>	-	0.00	1.79
	<u>27375</u>	-	-	0.00
t_i = loading time, t = desired time				

Table F.8. Shrinkage array for 2nd deck

Event	time index (t)	$\epsilon_{shD2}(t)$ ($\mu\epsilon$)
Time at which the deck is placed	<u>1</u>	0
Time at placement of barriers and the opening of bridge to traffic	<u>2</u>	0
Time at placement of overlay	<u>3</u>	0
Time at deck removal for projects that features a deck replacement	<u>4</u>	0
Time at new deck placement	<u>5</u>	0
Time at reopening of bridge to traffic	<u>6</u>	0
Time at placement of overlay	<u>7</u>	0
Time at the end of service life	<u>8</u>	580

- *Generate time and section dependent arrays for material and section properties*
- Modulus of elasticity based on AASHTO LRFD (2020) 9th edition with ACI 209R-92 development function

$$E_c(t) = \left(\frac{t}{\alpha + \beta t} \right)^{0.5} E_{c28} \quad \text{F-12}$$

$$E_{ci} = 120,000 K_1 w^{2.0} (f'_{ci})^{1/3} \quad \text{F-13}$$

$$E_{c28} = 120,000 K_1 w^{2.0} (f'_c)^{1/3} \quad \text{F-14}$$

- *For 1st and 2nd deck*

ACI 209R-92 development model with coefficients corresponding to Type I – Moist-cured concrete is going to be utilized.

$$\alpha = 4 \text{ \& } \beta = 0.85$$

$$E'_{c28} = 120,000 \cdot (1.0) \cdot (0.145)^2(4)^{1/3} = 4005 \text{ ksi}$$

$$E_{cD1}(t) = \left(\frac{t - 2}{4 + (0.85) \cdot (t - 2)} \right)^{0.5} \cdot 4005$$

$$E_{cD2}(t) = \left(\frac{t - 18260}{4 + (0.85) \cdot (t - 18260)} \right)^{0.5} \cdot 4005$$

Table F.9. Elastic modulus development for 1st and 2nd deck with time

Event	<i>time index (t)</i>	<i>E_{cD1}(t)</i> (ksi)	<i>E_{cD2}(t)</i> (ksi)
Time at which the deck is placed	<u>1</u>	-	-
Time at placement of barriers and the opening of bridge to traffic	<u>2</u>	3359	-
Time at placement of overlay	<u>3</u>	3359	-
Time at deck removal for projects that features a deck replacement	<u>4</u>	4344	-
Time at new deck placement	<u>5</u>	4344	-
Time at reopening of bridge to traffic	<u>6</u>	4344	3359
Time at placement of overlay	<u>7</u>	4344	3359
Time at the end of service life	<u>8</u>	4344	4344

- Modular ratio and transformed area

Since transformed section properties are utilized in Approach No. 6, the deck is transformed to girder by using the equation below. Since deck modulus of elasticity varies with time, the modular ratio will also vary with respect to time.

$$\eta(t) = \frac{E_{tr}(t)}{E_G} \quad \text{F-15}$$

$$A_{dtr} = A \cdot \eta(t) \quad \text{F-16}$$

- *For 1st and 2nd deck*

$$\eta_D(t) = \frac{E_{cD}(t)}{E_G}$$

$$A_{dtr}(t) = A_d \cdot \eta_d(t)$$

Table F.10. Modular ratio and transformed area for the 1st and 2nd deck

time index (t)	Modular ratio		Transformed deck area (in.²)	
	$\eta_{d1}(t)$	$\eta_{d2}(t)$	$A_{d1tr}(t)$	$A_{d2tr}(t)$
<u>1</u>	-	-	-	-
<u>2</u>	0.12	-	117	-
<u>3</u>	0.12	-	117	-
<u>4</u>	0.15	-	151	-
<u>5</u>	0.15	-	151	-
<u>6</u>	0.15	0.12	151	117
<u>7</u>	0.15	0.12	151	117
<u>8</u>	0.15	0.15	151	151

- *Transformed area for composite structures*

$$A_{ctr}(t) = A_G + A_{dtr}(t) \quad \text{F-17}$$

Table F.11. Net transformed area for the 1st and 2nd composite structures

$A_{ctr}(t, x) \text{ (in.2)}$		$A_{c1tr}(t)$	$A_{c2tr}(t)$
time index (t)	<u>1</u>	90	-
	<u>2</u>	207	-
	<u>3</u>	207	-
	<u>4</u>	241	-
	<u>5</u>	241	90
	<u>6</u>	241	207
	<u>7</u>	241	207
	<u>8</u>	241	241

- *Transformed moment of inertia for composite structure*

$$y_{dc} = \frac{s_d \cdot t_d \cdot (0.5 \cdot t_d + t_h) + 0.5 \cdot b_f \cdot t_h^2}{A_d} \quad \text{F-18}$$

$$y_{dbottom} = h + y_{dc} \quad \text{F-19}$$

$$y_c(t) = \frac{A_G \cdot y_{Gbottom} + A_{dtr}(t) \cdot y_{dbottom}}{A_{ctr}(t)} \quad \text{F-20}$$

$$I_d = \frac{s_d \cdot t_d^3 + b_f \cdot t_h^3}{12} + s_d \cdot t_d \cdot (y_{dc} - 0.5 \cdot t_d - t_h)^2 + b_f \cdot t_h \cdot (y_{dc} - 0.5 \cdot t_h)^2 \quad \text{F-21}$$

$$I_{dtr} = I_d \cdot \eta_{d1}(t) \quad \text{F-22}$$

$$I_c(t) = I_G + A_G \cdot (y_c(t) - y_{Gbottom})^2 + I_{dtr}(t) + A_{dtr}(t) \cdot (y_c(t) - y_{dbottom})^2 \quad \text{F-23}$$

Table F.12. Centroid of transformed composite structure with respect to bottom of girder

$y_c(t)$ (in.)		$y_{c1}(t)$	$y_{c2}(t)$
$time$ $index(t)$	<u>1</u>	35.5	-
	<u>2</u>	58.7	-
	<u>3</u>	58.7	-
	<u>4</u>	61.2	-
	<u>5</u>	61.2	35.5
	<u>6</u>	61.2	58.7
	<u>7</u>	61.2	58.7
	<u>8</u>	61.2	61.2

Table F.13. Transformed moment of inertia for the 1st and 2nd deck

time index (t)	Transformed moment of inertia (in.⁴)	
	I_{d1tr}	I_{d2tr}
<u>1</u>	-	-
<u>2</u>	788	-
<u>3</u>	788	-
<u>4</u>	1019	-
<u>5</u>	1019	-
<u>6</u>	1019	788
<u>7</u>	1019	788
<u>8</u>	1019	1019

Table F.14. Moment of inertia of transformed composite structure with 1st and 2nd deck

$I_c(t)$ (in. ⁴)		$I_{c1}(t)$	$I_{c2}(t)$
<i>time index (t)</i>	<u>1</u>	75840	-
	<u>2</u>	162050	-
	<u>3</u>	162050	-
	<u>4</u>	171630	-
	<u>5</u>	171630	75840
	<u>6</u>	171630	162050
	<u>7</u>	171630	162050
	8	171640	171630

- Simpson's 1/3rd rule for numerical integration

A general idea behind the Simpson's 1/3rd rule is explained briefly below.

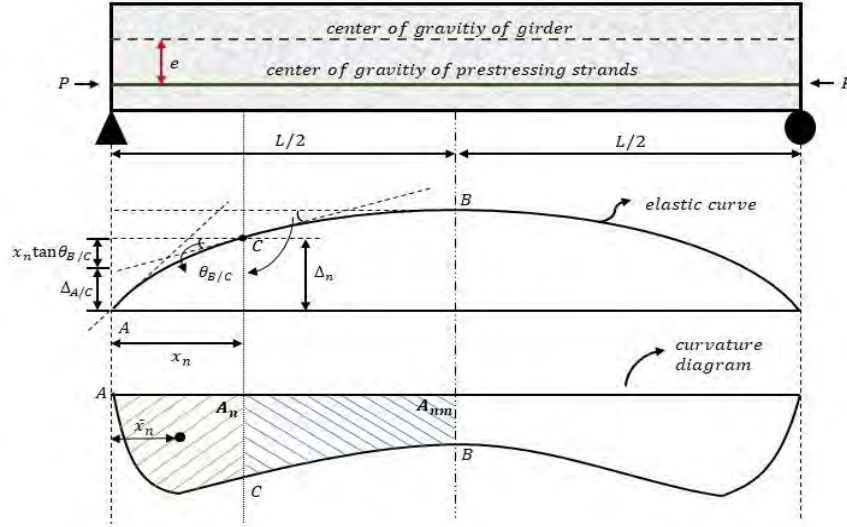


Figure F.3. Elastic curve and curvature diagram for typical prestressed concrete beam at release

$$\Delta_n = \Delta_{A/C} + x_n \tan \theta_{B/C} \quad \text{F-24}$$

$$\theta_{B/C} = A_{nm} \quad \text{F-25}$$

$$\Delta_{A/C} = A_n \bar{x}_n \quad \text{F-26}$$

$$\bar{x}_n = \frac{Q_n}{A_n} \quad \text{F-27}$$

$$Q_n = \frac{\Delta x}{3} (\phi_1 x_1 + 4\phi_2 x_2 + 2\phi_3 x_3 + 4\phi_4 x_4 + 2\phi_5 x_5 + \cdots + 4\phi_{n-1} x_{n-1} + \phi_n x_n) \quad \text{F-28}$$

$$A_n = \frac{\Delta x}{3} (\phi_1 + 4\phi_2 + 2\phi_3 + 4\phi_4 + 2\phi_5 + \cdots + 4\phi_{n-1} + \phi_n) \quad \text{F-29}$$

$$A_{nm} = \frac{\Delta x}{3} (\phi_n + 4\phi_{n+1} + 2\phi_{n+2} + \cdots + 4\phi_{m-1} + \phi_m) \quad \text{F-30}$$

$$\Delta x = \frac{L_G/2}{\text{number of sections} - 1} \quad \text{F-31}$$

- Time-dependent deflection calculations

Note: For both instantaneous and time-dependent deflection calculations, all matrices such as location, load effects, material properties and creep and shrinkage functions will be recalled using index number. In addition, steel girder itself does not creep or shrinkage, there is no time-dependent effect on camber and deflection.

- *For composite structure*

$$q = \text{rownumber}(N_D(t, x)) \quad \text{F-32}$$

$$m = \text{rownumber}(\text{loc}(x)) \quad \text{F-33}$$

$$\phi_p = \phi_0 \quad \text{F-34}$$

$$\Delta \phi_t(1, \text{loc}(x)) = 0 \quad \text{F-35}$$

$$y_{dG} = y_{dbottom} - y_{Gbottom} \quad \text{F-36}$$

$$i = \text{index}(t_i) \quad \text{F-37}$$

$$p = \text{index}(t_{i+1}) \quad \text{F-38}$$

$$r = i \quad \text{F-39}$$

$$k = [r; (r + 1); (r + 2); \dots (p - 2); (p - 1)] \quad \text{F-40}$$

- For each value in "k" array:

$$Co_{ND}(1, \text{loc}(x)) = 0 \ \& \ Co_{MD}(1, \text{loc}(x)) = 0 \quad \text{F-41}$$

$$\varepsilon_{shD} = -(\varepsilon_{d1}(k + 1) - \varepsilon_{d1}(k)) \quad \text{F-42}$$

$$z = [1; 2; \dots (q - 1); q] \quad \text{F-43}$$

- For each value in "z" array:

$$\psi_{crD} = \psi_D(t(k + 1), t(z)) - \psi_D(t(k), t(z)) \quad \text{F-44}$$

$$\psi_{crD,flex} = \psi_{crD} \cdot \beta_{flex} \quad \text{F-45}$$

$$E_D = E_{cD}(z) \quad \text{F-46}$$

$$h = [1; 2; \dots (m - 1); m] \quad \text{F-47}$$

- For each value in "h" array:

$$\text{if } N_D(z, h) > 0 \quad \psi_{crD,ax} = \psi_{crD} \cdot \beta_{ax} \quad \text{F-48}$$

$$\text{if } N_D(z, h) \leq 0 \quad \psi_{crD,ax} = \psi_{crD} \quad \text{F-49}$$

$$Co_{ND}(h) = \frac{N_D(z, h) \cdot \psi_{crD,ax}}{E_D \cdot A_D} + Co_{ND}(h) \quad \text{F-50}$$

$$Co_{MD}(h) = \frac{M_D(z, h) \cdot \psi_{crD,flex}}{E_D \cdot I_D} + Co_{MD}(h) \quad \text{F-51}$$

- End of "h" array.
- End of "z" array.

$$\psi_{crD,ax} = \psi_{crD} \quad \text{F-52}$$

$$stopper = 1 \quad \text{F-53}$$

$$j = [1; 2; \dots (m - 1); m] \quad \text{F-54}$$

- For each value in "j" array:

$$M_{co} = \begin{bmatrix} 1 & 1 & 0 & 0 & 0 & 0 & 0 \\ -y_{dG} & 0 & 1 & 1 & 0 & 0 & 0 \\ -\frac{1 + \psi_{crD,ax}}{E_D \cdot A_D} & 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & -\frac{1}{E_G \cdot A_G} & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & -\frac{1 + \psi_{crD,flex}}{E_D \cdot I_D} & 0 & 0 & 0 & 1 \\ 0 & 0 & 0 & -\frac{1}{E_G \cdot A_G} & 0 & 0 & 1 \\ 0 & 0 & 0 & 0 & 1 & -1 & y_{dG} \end{bmatrix} \quad \text{F-55}$$

$$M_c = \begin{bmatrix} 0 \\ 0 \\ Co_{ND}(j) + \varepsilon_{shD} \\ 0 \\ Co_{MD}(j) \\ 0 \\ 0 \end{bmatrix} \quad \text{F-56}$$

$$M_s = [M_{co}^{-1}] \cdot [M_c] \quad \text{F-57}$$

$$\begin{aligned} & \text{if } M_s(1) > 0 \\ & \quad \psi_{crD,ax} = \psi_{crD} \cdot \beta_{ax} \\ & \quad \text{stopper} = 0 \\ & \quad \text{continue} \end{aligned} \quad \text{F-58}$$

$$N_D(k+1, j) = M_s(1) \quad \text{F-59}$$

$$M_D(k+1, j) = M_s(3) \quad \text{F-60}$$

$$\Delta\phi_t(k+1, j) = M_s(7) + \Delta\phi_t(k, j) \quad \text{F-61}$$

$$\text{stopper} = 1 \quad \text{F-62}$$

$$\psi_{crd,ax} = \psi_{crD} \quad \text{F-63}$$

➤ End of "j" array.

$$\phi_t(k+1, loc(x)) = \phi_p + \Delta\phi_t(k+1, loc(x)) \quad F-64$$

➤ End of "k" array.

F.3. Pre-erection Camber

As it is stated in the problem definition, fabricated camber does not include the deflection caused by girder self-weight. Therefore, to be able to determine pre-erection camber, the girder self-weight effect needs to be calculated. Before, to be able to perform sectional analysis through the span, initial curvatures as well as camber at locations apart from midspan need to be calculated.

$$\alpha_2 = \frac{192 \cdot \Delta_{G0}}{5 \cdot L_G^4} \quad F-65$$

$$X_{G0f}(x) = \alpha_2 \cdot (x) \cdot (L_G - x) \quad F-66$$

Using fabricated curvature along the span with Eqns. between F-24 and F-31, fabricated camber can be determined. It should be noted that although the fabricated camber along the span could be calculated exactly without using numerical integration, it is preferred to be calculated via this approach to get consistent results. If sufficiently enough section is considered, this would not yield significant error in the calculations.

Table F.15. Fabricated camber and curvature along the span

$\Delta_{G0f}(x)$ (in.)			$X_{G0f}(x)$ (1/in.)		
$\Delta_{G0f}(1)$	$\Delta_{G0f}(2)$	$\Delta_{G0f}(3)$	$X_{G0f}(1)$	$X_{G0f}(2)$	$X_{G0f}(3)$
0.00	-7.32	-14.64	0.00	-2.45E-05	-3.26E-05

➤ Girder Self-weight Effect

$$w_G = \gamma_G \cdot A_G \quad F-67$$

$$\text{if } x \leq L_b \quad M_{Gsw}(x) = -\frac{w_G \cdot x^2}{2} \quad F-68$$

$$\text{if } x > L_b \quad M_{Gsw}(x) = \frac{w_G \cdot L_G \cdot (x - L_b)}{2} - \frac{w_G \cdot x^2}{2} \quad \text{F-69}$$

$$\Delta\phi_{Gsw}(x) = \frac{M_{Gsw}(x)}{E_G \cdot I_G} \quad \text{F-70}$$

$$w_G = \frac{0.490 \cdot 90}{12^3} = 0.026 \text{ kips/in.}$$

Using Eqns. F-67, F-68 and F-69, moment and change in curvature due to girder self-weight along the span can be calculated as shown in Table F.16.

Table F.16. Moment and change in curvature due to girder self-weight along the span

$M_{Gsw}(x) \text{ (kips} \cdot \text{in.)}$			$\Delta\phi_{Gsw}(x) \text{ (1/in.)}$		
$M_{Gsw}(1)$	$M_{Gsw}(2)$	$M_{Gsw}(3)$	$\Delta\phi_{Gsw}(1)$	$\Delta\phi_{Gsw}(2)$	$\Delta\phi_{Gsw}(3)$
0	9677	13113	0	4.48E-06	6.07E-06

Net curvature after girder erection can be calculated by simply summing the curvature values in Table F.15 and Table F.16, as provided in Table F.17.

Table F.17. Net curvature after girder erection

$\phi_{pre}(x) \text{ (1/in.)}$		
$\phi_{pre}(1)$	$\phi_{pre}(2)$	$\phi_{pre}(3)$
0	-2.00E-05	-2.65E-05

Using the Eqns. between F-24 and F-31, camber at considered sections can be calculated as follows:

$$\Delta x = \frac{\frac{173 \cdot 12}{2}}{3 - 1} = 519 \text{ in.}$$

$$A_{nm}(1) = \frac{519}{3} \cdot [0 + 4 \cdot (-2.00 \cdot 10^{-5}) + (-2.65 \cdot 10^{-5})] = -18.42 \cdot 10^{-3} \text{ in.}^2$$

$$A_{nm}(2) = \frac{519}{3} \cdot [(-2.00 \cdot 10^{-5}) + (-2.65 \cdot 10^{-5})] = -8.04 \cdot 10^{-3} \text{ in.}^2$$

$$A_{nm}(3) = 0$$

$$A_n(1) = 0$$

$$A_n(2) = \frac{519}{3} \cdot [0 + (-2.00 \cdot 10^{-5})] = -3.46 \cdot 10^{-3} \text{ in.}^2$$

$$A_n(3) = \frac{519}{3} \cdot [0 + 4 \cdot (-2.00 \cdot 10^{-5}) + (-2.65 \cdot 10^{-5})] = -18.42 \cdot 10^{-3} \text{ in.}^2$$

$$Q_n(1) = 0$$

$$Q_n(2) = \frac{519}{3} \cdot [0 + (-2.00 \cdot 10^{-5}) \cdot 519] = -1.80 \text{ in.}^3$$

$$Q_n(3) = \frac{519}{3} \cdot [0 + 4 \cdot (-2.00 \cdot 10^{-5}) \cdot 519 + (-2.65 \cdot 10^{-5}) \cdot 1038] = -11.94 \text{ in.}^3$$

Table F.18. Application of Simpson's rule and pre-erection camber

Parameters	<i>Section index (x)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>
$Q_n(x) \text{ (in.}^3\text{)}$	0	-1.80	-11.94
$A_n(x) \text{ (in.}^2\text{)}$	0	-3.46E-03	-1.84E-02
$\bar{x}_n(x) \text{ (in.)}$	0	520.23	648.91
$\Delta_{A/C}(x) \text{ (in.)}$	0	-1.80	-11.94
$\theta_{B/C}(x) \text{ (rad.)}$	-1.84E-02	-8.04E-03	0
$x_n(x) \text{ (in.)}$	0	519.00	1038.00
$\Delta_{pre}(x) \text{ (in.)}$	0	-5.97	-11.94

F.4. Camber just after Placement of 1st Deck

- Deck self-weight effect

$$w_{d1} = A_{d1} \cdot \gamma_{d1} = \frac{990.36 \cdot 0.150}{12^3} = 0.086 \text{ kips/in.}$$

if $x \leq 24$

$$M_{d1sw}(x) = -0.043 \cdot x^2$$

$$\text{if } 24 < x \quad M_{d1sw}(x) = 87.2 \cdot x - 0.043 \cdot (x - 24)^2 - 2117.5 \quad \text{F-72}$$

$$\Delta\phi_{d1sw}(x) = \frac{M_{d1sw}(x)}{E_G \cdot I_G} \quad \text{F-73}$$

Table F.19. Moment and change in curvature due to 1st deck self-weight

$M_{d1sw}(x)$ (kips – in.)			$\Delta\phi_{d1sw}(x)$ (1/in.)		
$M_{d1sw}(1)$	$M_{d1sw}(2)$	$M_{d1sw}(3)$	$\Delta\phi_{d1sw}(1)$	$\Delta\phi_{d1sw}(2)$	$\Delta\phi_{d1sw}(3)$
0	32593	44172	0	1.51E-05	2.04E-05

To obtain the net curvature just after 1st deck is placed, the net curvature values from previous calculation, Table F.17, should be summed up with the change in curvature values given in Table F.19.

Table F.20. Net curvature just after 1st deck is placed

$\phi_{ad1p}(x)$ (1/in.)		
$\phi_{ad1p}(1)$	$\phi_{ad1p}(2)$	$\phi_{ad1p}(3)$
0	-4.90E-06	-6.11E-06

- Calculation of camber just after placement of 1st deck

Using eqns. between F-24 and F-31 with the net curvature values given in Table F.20, the camber just after 1st deck is placed can be estimated.

Table F.21. Camber just after placement of 1st deck

Camber (in.)	section index (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nad1p}(x)$	0	-1.43	-2.86

F.5. Camber just before Application of Barrier and Overlay Loads on 1st Superstructure

In this part, a time-dependent analysis for composite structure is conducted. Since steel girder does not creep or shrink, only the force history of deck is considered. In this step, there is no force on the deck.

Using Eqns. between F-24 and F-64, the time-dependent calculation can be performed time t_1 and t_2 .

$$m = 3 \text{ \& } q = 1$$

$$\phi_p(x) = [0 \quad -5.01 \cdot 10^{-6} \quad -6.25 \cdot 10^{-6}]$$

$$\Delta\phi_t(x) = [0; 0; 0]$$

$$y_{dG} = 41 \text{ in.}$$

$$i = 1 \text{ \& } p = 2 \text{ \& } r = 1$$

$$k = [1]$$

- For $k = 1$

$$Co_{ND}(1, loc(x)) = 0 \text{ \& } Co_{MD}(1, loc(x)) = 0$$

$$\varepsilon_{shD} = -(\varepsilon_{d1}(2) - \varepsilon_{d1}(1)) = 0$$

$$z = [1]$$

- For $z = 1$

$$\psi_{crD} = \psi_D(t(2), t(1)) - \psi_D(t(1), t(1)) = 0$$

$$\psi_{crD, flex} = \psi_{crD}$$

$$E_D = 0$$

$$h = [1; 2; 3]$$

$$Co_{ND} = [0; 0; 0]$$

$$Co_{MD} = [0; 0; 0]$$

- End of "z" array.

$$j = [1; 2; 3]$$

- For $j = 1$

$$M_{co} = \begin{bmatrix} 1 & 1 & 0 & 0 & 0 & 0 & 0 \\ -41 & 0 & 1 & 1 & 0 & 0 & 0 \\ -10^{20} & 0 & 0 & 0 & 1 & 0 & 0 \\ 0 & -3.90 \cdot 10^{-7} & 0 & 0 & 0 & 1 & 0 \\ 0 & 0 & -10^{20} & 0 & 0 & 0 & 1 \\ 0 & 0 & 0 & -4.63 \cdot 10^{-10} & 0 & 0 & 1 \\ 0 & 0 & 0 & 0 & 1 & -1 & 41 \end{bmatrix}$$

$$M_c = \begin{bmatrix} 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \\ 0 \end{bmatrix}$$

$$M_s = [M_{co}^{-1}] \cdot [M_c]$$

$$N_D(2,1) = 0$$

$$M_D(2,1) = 0$$

$$\Delta\phi_t(2,1) = 0$$

Repeat the same procedure for other sections: $j = 2$ and $j = 3$. Basically, since both creep coefficient and shrinkage strain values are 0 in this step, there will be no change in curvature thus in camber.

➤ End of "j" array.

$$\phi_t(2, loc(x)) = \phi_p + \Delta\phi_t(2, loc(x))$$

$$\phi_t(2, loc(x)) = \phi_p = [0 \quad -4.90 \cdot 10^{-6} \quad -6.11 \cdot 10^{-6}]$$

➤ End of "k" array.

Since there is no change in curvature in this time step, previous camber remains as it is.

Table F.22. Camber just before application of barrier and overlay loads on 1st superstructure

Camber (in.)	<i>section index (x)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nbas1}(x)$	0	-1.43	-2.86

F.6. Camber just after Application of Barrier and Overlay Loads on 1st Superstructure

As it is indicated above, it is assumed that barrier and overlay loads are applied simultaneously on the 1st composite structure. In this part, only instantaneous changes in camber due to application of superimposed dead loads that are barrier and overlay loads are calculated. In other words, no time-dependent calculation is going to be performed.

- Barrier load effect

$$w_{b1} = 0.04 \text{ kips/in.}$$

$$\text{if } x \leq 24 \quad M_{b1sw}(x) = -0.02 \cdot x^2 \quad \text{F-74}$$

$$\text{if } 24 < x \quad M_{b1sw}(x) = 40.56 \cdot x - 0.02 \cdot (x - 24)^2 - 984.96 \quad \text{F-75}$$

$$\Delta\phi_{b1sw}(x) = \frac{M_{b1sw}(x)}{E_G \cdot I_{c1}(2)} \quad \text{F-76}$$

- Overlay load effect

$$w_{o1} = t_{o1} \cdot s_d \cdot \gamma_{o1} = \frac{2 \cdot 110.04 \cdot 0.140}{12^3} = 0.0178 \text{ kips/in.}$$

$$\text{if } x \leq 24 \quad M_{o1sw}(x) = -8.92 \cdot 10^{-3} \cdot x^2 \quad \text{F-77}$$

$$\text{if } 24 < x \quad M_{o1sw}(x) = 18.08 \cdot x - 8.92 \cdot 10^{-3} \cdot (x - 24)^2 - 439.06 \quad \text{F-78}$$

$$\Delta\phi_{o1sw}(x) = \frac{M_{o1sw}(x)}{E_G \cdot I_{c1}(3)} \quad \text{F-79}$$

Table F.23. Moment applied on composite structure due to barrier and overlay loads

$M_{b1sw}(x) \text{ (kips} \cdot \text{in.)}$			$M_{o1sw}(x) \text{ (kips} \cdot \text{in.)}$		
$M_{b1sw}(1)$	$M_{b1sw}(2)$	$M_{b1sw}(3)$	$M_{o1sw}(1)$	$M_{o1sw}(2)$	$M_{o1sw}(3)$
0	15165	20552	0	6760	9162

Table F.24. Change in curvature due to barrier and overlay load

$\Delta\phi_{b1sw}(x) \text{ (1/in.)}$			$\Delta\phi_{o1sw}(x) \text{ (1/in.)}$		
$\Delta\phi_{b1sw}(1)$	$\Delta\phi_{b1sw}(2)$	$\Delta\phi_{b1sw}(3)$	$\Delta\phi_{o1sw}(1)$	$\Delta\phi_{o1sw}(2)$	$\Delta\phi_{o1sw}(3)$
0	3.28E-06	4.45E-06	0	1.46E-06	1.98E-06

- Calculation of camber just after application of barrier and overlay loads

By summing the net curvature values from previous part, Table F.20, with the change in curvature values due to barrier and overlay loads, Table F.24, the net curvature can be obtained as shown in Table F.25. Then, using eqns. between F-24 and F-31, the camber just after application of barrier and overlay loads can be estimated as shown in Table F.26. Actually, zero curvature values mean that the girder is flat that is there is no camber and deflection in the girder just after application of barrier and overlay load.

Table F.25. Net curvature just after application of barrier and overlay loads

$\phi_{naas1}(x) \text{ (1/in.)}$		
$\phi_{naas1}(1)$	$\phi_{naas1}(2)$	$\phi_{naas1}(3)$
0	0	0

Table F.26. Camber just after application of barrier and overlay loads

Camber (in.)	<i>section index (x)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{naas1}(x) \text{ (in.)}$	0	0	0

Please note that there no camber or deflection on the girder just after application of barrier and overlay loads. This was done on purpose such that the fabricated camber value was adjusted accordingly.

F.7. Camber just before Deck Removal Process

In this part, a time-dependent analysis for composite structure is conducted. To do so, the force history array for deck should be updated by considering the change in forces on the deck due

to previous time-dependent effects between t_1 and t_2 (which is zero in this case), barrier and overlay self-weight.

- Time-dependent effect between t_1 and t_2

In this case, since only one time step is considered between t_1 and t_2 , there is no force generated on the deck.

- Barrier load effect

$$\Delta N_{Gb1sw}(x) = \frac{M_{b1sw}(x) \cdot A_G \cdot [y_{c1}(2) - y_{Gbottom}]}{I_{c1}(2)} \quad \text{F-80}$$

$$\Delta M_{Gb1sw}(x) = \frac{M_{b1sw}(x) \cdot I_G}{I_{c1}(2)} \quad \text{F-81}$$

$$\Delta N_{d1b1sw}(x) = \frac{M_{b1sw}(x) \cdot A_{d1tr}(2) \cdot [y_{c1}(2) - y_{d1bottom}]}{I_{c1}(2)} \quad \text{F-82}$$

$$\Delta M_{d1b1sw}(x) = \frac{M_{b1sw}(x) \cdot I_{d1tr}(2)}{I_{c1}(2)} \quad \text{F-83}$$

Table F.27. Change in forces on deck due to barrier load

$\Delta N_{d1b1sw}(\text{kips})$			$\Delta M_{d1b1sw}(\text{kips} - \text{in.})$		
$\Delta N_{d1b1sw}(1)$	$\Delta N_{d1b1sw}(2)$	$\Delta N_{d1b1sw}(3)$	$\Delta M_{d1b1sw}(1)$	$\Delta M_{d1b1sw}(2)$	$\Delta M_{d1b1sw}(3)$
0	-195	-264	0	73	100

- Overlay load effect

$$\Delta N_{Go1sw}(x) = \frac{M_{o1sw}(x) \cdot A_G \cdot [y_{c1}(3) - y_{Gbottom}]}{I_{c1}(3)} \quad \text{F-84}$$

$$\Delta M_{Go1sw}(x) = \frac{M_{o1sw}(x) \cdot I_G}{I_{c1}(3)} \quad \text{F-85}$$

$$\Delta N_{d1o1sw}(x) = \frac{M_{o1sw}(x) \cdot A_{d1tr}(3) \cdot [y_{c1}(3) - y_{d1bottom}]}{I_{c1}(3)} \quad \text{F-86}$$

$$\Delta M_{d1o1sw}(x) = \frac{M_{o1sw}(x) \cdot I_{d1tr}(3)}{I_{c1}(3)} \quad \text{F-87}$$

Table F.28. Change in forces on deck due to overlay load

$\Delta N_{d1o1sw} (kips)$			$\Delta M_{d1o1sw} (kips - in.)$		
$\Delta N_{d1o1sw}(1)$	$\Delta N_{d1o1sw}(2)$	$\Delta N_{d1o1sw}(3)$	$\Delta M_{d1o1sw}(1)$	$\Delta M_{d1o1sw}(2)$	$\Delta M_{d1o1sw}(3)$
0	-87	-118	0	33	45

By summing up the change in forces for deck due to abovementioned effects, the force history array for deck can be generated as shown in Table F.29.

Table F.29. Net force history array just after application of barrier and overlay loads

<i>index</i>	$N_D(t, x) (kips)$			$M_D(t, x) (kips - in.)$		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>1</u>	0	0	0	0	0	0
<u>2</u>	0	-195	-264	0	73	100
<u>3</u>	0	-87	-118	0	33	45

It should be noted that although there is no time-dependent change in forces between t_2 and t_3 , an additional row is added for this in Table F.29. This is done on purpose so that the time indexes match with the *MDOTCamber* program where the effects of barrier and overlay loads are considered separately.

After obtaining the force history array for the deck, using Eqns. between F-24 and F-31, time-dependent calculation can be performed between time t_3 and t_4 . Since it is done explicitly in section F.5, it is not repeated here.

After conducting time-dependent calculation, the following change in force and curvature values can be obtained.

Table F.30. Change in forces on deck due to time-dependent effects between t_3 and t_4

$\Delta N_{Dt_3t_4} (kips)$			$\Delta M_{Dt_3t_4} (kips - in.)$		
$\Delta N_{Dt_3t_4}(1)$	$\Delta N_{Dt_3t_4}(2)$	$\Delta N_{Dt_3t_4}(3)$	$\Delta M_{Dt_3t_4}(1)$	$\Delta M_{Dt_3t_4}(2)$	$\Delta M_{Dt_3t_4}(3)$
290	365	392	44	-13	-33

Table F.31. Change in curvature between time t_3 and t_4

$\Delta\phi_t(x) \text{ (1/in.)}$		
$\Delta\phi_t(1)$	$\Delta\phi_t(2)$	$\Delta\phi_t(3)$
5.48E-06	6.93E-06	7.45E-06

Using the previous net curvature values given in Table F.25, and change in curvature values in Table F.31, net curvature values just before the deck removal process can be obtained as follows:

Table F.32. Net curvature just before deck removal process

$\phi_t(4,x) \text{ (1/in.)}$		
$\phi_t(4,1)$	$\phi_t(4,2)$	$\phi_t(4,3)$
5.48E-06	6.78E-06	7.78E-06

Using net curvature values provided in Table F.32 and Eqns. between F-24 and F-31, camber just before the deck removal process can be calculated as shown in Table F.33.

Table F.33. Camber just before deck removal process

Camber (in.)	section index (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nbd1r}(x)$	0	1.92	3.83

F.8. Camber just after Deck Removal Process

Instantaneous change in camber just after deck is removed is composed of two parts: one is due to removal of self-weight loads of deck, overlay and barrier; other is due to effect of locked forces inside the deck on girder.

- Removal of barrier and overlay loads

$$\Delta N_{d1ob1r}(x) = - \frac{[M_{b1sw}(x) + M_{o1sw}(x)] \cdot A_{d1tr}(4) \cdot [y_{c1}(4) - y_{d1bottom}]}{I_{c1}(4)} \quad \text{F-88}$$

$$\Delta M_{d1ob1r}(x) = - \frac{[M_{b1sw}(x) + M_{o1sw}(x)] \cdot I_{d1tr}(4)}{I_{c1}(4)} \quad \text{F-89}$$

$$\Delta\phi_{ob1r}(x) = - \frac{[M_{b1sw}(x) + M_{o1sw}(x)]}{E_G \cdot I_{c1}(4)} \quad \text{F-90}$$

Table F.34. Change in forces on deck due to removal of barrier and overlay loads

$\Delta N_{d1ob1r}(x) \text{ (kips)}$			$\Delta M_{d1ob1r}(x) \text{ (kips} - \text{in.)}$		
$\Delta N_{d1ob1r}(1)$	$\Delta N_{d1ob1r}(2)$	$\Delta N_{d1ob1r}(3)$	$\Delta M_{d1ob1r}(1)$	$\Delta M_{d1ob1r}(2)$	$\Delta M_{d1ob1r}(3)$
0	295	400	0	-130	-176

Table F.35. Change in curvature due to removal of barrier and overlay loads

$\Delta \phi_{ob1r}(x) \text{ (1/in.)}$		
$\Delta \phi_{ob1r}(1)$	$\Delta \phi_{ob1r}(2)$	$\Delta \phi_{ob1r}(3)$
0	-4.48E-06	-6.07E-06

- Removal of locked forces in the deck before it is removed

Using Table F.29, Table F.30 and Table F.34, the total net forces on the deck can be calculated as shown in Table F.36.

Table F.36. Net forces on the deck just before it is removed

$N_{d1br} \text{ (kips)}$			$M_{d1br} \text{ (kips} - \text{in.)}$		
$N_{d1br}(1)$	$N_{d1br}(2)$	$N_{d1br}(3)$	$M_{d1br}(1)$	$M_{d1br}(2)$	$M_{d1br}(3)$
290	379	410	44	-36	-65

After determined the net forces on the deck, their effects on steel girder is considered when estimating the camber using the equations below. It should be noted that only change in moment would be sufficient to calculate change in curvature due to removal of locked forces inside the deck.

$$\Delta M_{Gd1r}(x) = M_{d1br}(x) + N_{d1br}(x) \cdot [y_{Gbottom} - y_{d1bottom}] \quad \text{F-91}$$

$$\Delta \phi_{d1r}(x) = \frac{\Delta M_{Gd1r}(x)}{E_G \cdot I_G} \quad \text{F-92}$$

Table F.37. Change in moment due to removal of locked forces inside the deck

$\Delta M_{Gd1r}(x) \text{ (kips} - \text{in.)}$		
$\Delta M_{Gd1r}(1)$	$\Delta M_{Gd1r}(2)$	$\Delta M_{Gd1r}(3)$
-11849	-15561	-16879

Table F.38. Change in curvature due to removal of locked forces inside the deck

$\Delta\phi_{d1r}(x) \text{ (1/in.)}$		
$\Delta\phi_{d1r}(1)$	$\Delta\phi_{d1r}(2)$	$\Delta\phi_{d1r}(3)$
-5.48E-06	-7.19E-06	-7.81E-06

- Removal of deck load

$$\Delta M_{Gd1swr}(x) = -M_{d1sw}(x) \quad \text{F-93}$$

$$\Delta\phi_{d1swr}(x) = \frac{\Delta M_{Gd1swr}(x)}{E_G \cdot I_G} \quad \text{F-94}$$

Table F.39. Change in curvature due to deck removal

$\Delta\phi_{d1swr}(x) \text{ (1/in.)}$		
$\Delta\phi_{d1swr}(1)$	$\Delta\phi_{d1swr}(2)$	$\Delta\phi_{d1swr}(3)$
0	-1.51E-05	-2.04E-05

After considered all effects, curvature changes given in Table F.35, Table F.38 and Table F.39 need to be summed together with the previous net curvature values provided in Table F.32 so that the net curvature just after deck removal can be obtained as shown in Table F.40.

Table F.40. Net curvature just after deck removal

$\phi_{ad1r}(x) \text{ (1/in.)}$		
$\phi_{ad1r}(1)$	$\phi_{ad1r}(2)$	$\phi_{ad1r}(3)$
0	-2.00E-05	-2.65E-05

Using net curvature values provided in Table F.40 and Eqns. between F-24 and F-31, camber just after the deck is removed can be calculated as shown in Table F.41.

Table F.41. Camber just after deck removal

Camber (in.)	<i>section index</i> (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nad1r}(x)$	0	-5.97	-11.94

It should be noted here that the net curvature and camber values given in Table F.40 and Table F.41, respectively, are same with the curvature and camber values just after girder is erected,

Table F.17 and Table F.18. Basically, it means that after the deck is removed, girder turns back to its original position, and this is expected because steel girder itself does not creep or shrink.

F.9. Camber just before 2nd Deck Placement

In this part, since steel girder does not creep or shrinkage, no time-dependent analysis is conducted, and the net curvature and camber values provided in Table F.40 and Table F.41 remains as they are, respectively.

Table F.42. Net curvature just before new deck placement

$\phi_{nbndp}(x)$ (1/in.)		
$\phi_{nbndp}(1)$	$\phi_{nbndp}(2)$	$\phi_{nbndp}(3)$
0	-2.00E-05	-2.65E-05

Table F.43. Camber just before new deck placement

Camber (in.)	<i>section index</i> (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nbndp}(x)$	0	-5.97	-11.94

F.10. Camber just after 2nd Deck Placement

A similar process that is explained in section F.4 is followed in this part to estimate the camber. Since it is assumed that 1st and 2nd decks are identical, equations used for 1st deck are valid for 2nd deck.

- Deck self-weight effect

$$w_{d2} = A_{d2} \cdot \gamma_{d2} = \frac{990.36 \cdot 0.150}{12^3} = 0.086 \text{ kips/in.}$$

$$\text{if } x \leq 24 \quad M_{d2sw}(x) = -0.043 \cdot x^2 \quad \text{F-95}$$

$$\text{if } 24 < x \quad M_{d2sw}(x) = 87.2 \cdot x - 0.043 \cdot (x - 24)^2 - 2117.5 \quad \text{F-96}$$

$$\Delta\phi_{d2sw}(x) = \frac{M_{d2sw}(x)}{E_G \cdot I_G} \quad \text{F-97}$$

Table F.44. Moment and change in curvature due to 2nd deck self-weight

$M_{d2sw}(x) \text{ (kips} - \text{in.)}$			$\Delta\phi_{d2sw}(x) \text{ (1/in.)}$		
$M_{d2sw}(1)$	$M_{d2sw}(2)$	$M_{d2sw}(3)$	$\Delta\phi_{d2sw}(1)$	$\Delta\phi_{d2sw}(2)$	$\Delta\phi_{d2sw}(3)$
0	32593	44172	0.00	1.51E-05	2.04E-05

To obtain the net curvature just after 2nd deck is placed, the net curvature values from previous step, Table F.42, should be summed up with the change in curvature values given in Table F.44.

Table F.45. Net curvature just after 2nd deck is placed

$\phi_{ad2p}(x) \text{ (1/in.)}$		
$\phi_{ad2p}(1)$	$\phi_{ad2p}(2)$	$\phi_{ad2p}(3)$
0	-4.90E-06	-6.11E-06

- Calculation of camber just after placement of 2nd deck

Using Eqns. between F-24 and F-31 with the net curvature values given in Table F.45, the camber just after 1st deck is placed can be estimated.

Table F.46. Camber just after placement of 2nd deck

Camber (in.)	$\text{section index } (x)$		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nad2p}(x)$	0	-1.43	-2.86

F.11. Camber just before Application of Barrier and Overlay Loads on 2nd Superstructure

In this part, a time-dependent analysis for composite structure is conducted similar to section F.5. Since steel girder does not creep or shrink, only the force history of deck is considered. In this step, since the deck is placed recently, there is no force on the deck.

Using Eqns. between F-32 and F-64, the time-dependent calculation for 2nd composite structure between time t_5 and t_6 can be performed. As it is done in section F.5, it will not be repeated here, but in any way, since creep coefficient and shrinkage strain values for 2nd deck are zero in this time step, there will be no time-dependent change in curvature, thus in camber, at the

end of time step. Therefore, net curvature and camber values in Table F.45 and Table F.46 remains as they are, respectively.

Table F.47. Net curvature just before application of barrier and overlay loads

$\phi_t(6, x) \text{ (1/in.)}$		
$\phi_t(6,1)$	$\phi_t(6,2)$	$\phi_t(6,3)$
0	-4.90E-06	-6.11E-06

Table F.48. Camber just before application of barrier and overlay loads on 2nd superstructure

Camber (in.)	<i>section array</i> (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{nbas2}(x)$	0	-1.43	-2.86

F.12. Camber just after Application of Barrier and Overlay Loads on 2nd Superstructure

Similarly, the process provided in section F.6, is followed in this part to estimate the camber just after application of barrier and overlay loads on 2nd super-structure. Since it is assumed that same barrier and overlay loads are applied on it, equations used previously for the effect of barrier and overlay loads on 1st super-structure are valid for 2nd super-structure, however, parameters that are time-dependent should be updated accordingly. Similarly, it is assumed that overlay load is simultaneously applied with the barrier load for simplicity, hence no time-dependent calculation is going to be performed between application of barrier and overlay loads.

- Barrier load effect

$$w_{b2} = 0.04 \text{ kips/in.}$$

$$\text{if } x \leq 24 \quad M_{b2sw}(x) = -0.02 \cdot x^2 \quad \text{F-98}$$

$$\text{if } 24 < x \quad M_{b2sw}(x) = 40.56 \cdot x - 0.02 \cdot (x - 24)^2 - 984.96 \quad \text{F-99}$$

$$\Delta\phi_{b2sw}(x) = \frac{M_{b2sw}(x)}{E_G \cdot I_{c2}(6)} \quad \text{F-100}$$

- Overlay load effect

$$w_{o2} = t_{o2} \cdot s_d \cdot \gamma_{o2} = \frac{2 \cdot 110.04 \cdot 0.140}{12^3} = 0.0178 \text{ kips/in.}$$

$$\text{if } x \leq 24 \quad M_{o2sw}(x) = -8.92 \cdot 10^{-3} \cdot x^2 \quad \text{F-101}$$

$$\text{if } 24 < x \quad M_{o2sw}(x) = 18.08 \cdot x - 8.92 \cdot 10^{-3} \cdot (x - 24)^2 - 439.06 \quad \text{F-102}$$

$$\Delta\phi_{o2sw}(x) = \frac{M_{o2sw}(x)}{E_G \cdot I_{c2}(7)} \quad \text{F-103}$$

Table F.49. Moment applied on composite structure due to barrier and overlay loads

$M_{b2sw}(x) \text{ (kips} \cdot \text{in.)}$			$M_{o2sw}(x) \text{ (kips} \cdot \text{in.)}$		
$M_{b2sw}(1)$	$M_{b2sw}(2)$	$M_{b2sw}(3)$	$M_{o2sw}(1)$	$M_{o2sw}(2)$	$M_{o2sw}(3)$
0	15165	20552	0	6760	9162

Table F.50. Change in curvature due to barrier and overlay load

$\Delta\phi_{b2sw}(x) \text{ (1/in.)}$			$\Delta\phi_{o2sw}(x) \text{ (1/in.)}$		
$\Delta\phi_{b2sw}(1)$	$\Delta\phi_{b2sw}(2)$	$\Delta\phi_{b2sw}(3)$	$\Delta\phi_{o2sw}(1)$	$\Delta\phi_{o2sw}(2)$	$\Delta\phi_{o2sw}(3)$
0	3.28E-06	4.45E-06	0	1.46E-07	1.98E-06

- Calculation of camber just after application of barrier and overlay loads

By summing the net curvature values from previous part, Table F.47, with the change in curvature values due to barrier and overlay loads, Table F.50, the net curvature can be obtained as shown in Table F.51. Then, using Eqns. between F-24 and F-31, the camber just after application of barrier and overlay loads on 2nd super-structure can be estimated as shown in Table F.52.

Table F.51. Net curvature just after application of barrier and overlay loads on 2nd super-structure

$\phi_{naas2}(x) \text{ (1/in.)}$		
$\phi_{naas1}(1)$	$\phi_{naabo1}(2)$	$\phi_{naabo1}(3)$
0	0	0

Table F.52. Camber just after application of barrier and overlay loads on 2nd super-structure

Camber (in.)	section index (x)		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{naas2}(x) \text{ (in.)}$	0	0	0

F.13. Camber at Final Time

In this part, a time-dependent analysis for composite structure is conducted. To do so, the force history array for deck should be generated by considering the change in forces on the deck due to previous time-dependent effects between t_5 and t_6 (which is zero in this case), barrier and overlay self-weight. A similar approach stated in section F.7 is followed. Equations used in section F.7 are valid in this part, however, parameters that are time-dependent should be updated accordingly.

- Time-dependent effect between t_5 and t_6

In this case, since only one time step is considered between t_5 and t_6 , there is no force generated on the deck.

- Barrier load effect

$$\Delta N_{d2b2sw}(x) = \frac{M_{b2sw}(x) \cdot A_{d2tr}(6) \cdot [y_{c2}(6) - y_{d2bottom}]}{I_{c2}(6)} \quad \text{F-104}$$

$$\Delta M_{d2b2sw}(x) = \frac{M_{b2sw}(x) \cdot I_{d2tr}(6)}{I_{c2}(6)} \quad \text{F-105}$$

Table F.53. Change in forces on deck due to barrier load

$\Delta N_{d2b2sw}(kips)$			$\Delta M_{d2b2sw}(kips - in.)$		
$\Delta N_{d2b2sw}(1)$	$\Delta N_{d2b2sw}(2)$	$\Delta N_{d2b2sw}(3)$	$\Delta M_{d2b2sw}(1)$	$\Delta M_{d2b2sw}(2)$	$\Delta M_{d2b2sw}(3)$
0	-195	-264	0	74	100

- Overlay load effect

$$\Delta N_{d2o2sw}(x) = \frac{M_{o2sw}(x) \cdot A_{d2tr}(7) \cdot [y_{c2}(7) - y_{d2bottom}]}{I_{c2}(3)} \quad \text{F-106}$$

$$\Delta M_{d2o2sw}(x) = \frac{M_{o2sw}(x) \cdot I_{d2tr}(7)}{I_{c2}(7)} \quad \text{F-107}$$

Table F.54. Change in forces on deck due to overlay load

$\Delta N_{d2o2sw}(kips)$			$\Delta M_{d2o2sw}(kips - in.)$		
$\Delta N_{d2o2sw}(1)$	$\Delta N_{d2o2sw}(2)$	$\Delta N_{d2o2sw}(3)$	$\Delta M_{d2o2sw}(1)$	$\Delta M_{d2o2sw}(2)$	$\Delta M_{d2o2sw}(3)$
0	-87	-118	0	33	45

By summing up the change in forces for deck due to abovementioned effects, the force history array for deck can be generated as shown in Table F.55. Please note that the values in force history array for the 2nd deck for the indices from 1 to 5 are taken as 0 because the 2nd is cast after time index 5.

Table F.55. Net force history array just after application of barrier and overlay loads

<i>index</i>	$N_D(t, x) (kips)$			$M_D(t, x) (kips - in.)$		
	<u>1</u>	<u>2</u>	<u>3</u>	<u>1</u>	<u>2</u>	<u>3</u>
<u>5</u>	0	0	0	0	0	0
<u>6</u>	0	-195	-264	0	74	100
<u>7</u>	0	-87	-118	0	33	45

It should be noted that although there is no time-dependent change in forces between t_6 and t_7 , an additional row is added for this in Table F.55. This is done on purpose so that the time indices match with the *MDOTCamber* program where the effects of barrier and overlay loads are considered separately.

After obtaining the force history array for the 2nd deck, using Eqns. between F-24 and F-64, time-dependent calculation can be performed between time t_7 and t_8 . Since it is done explicitly in section F.7, it is not repeated here.

After conducting time-dependent calculation, the following change in force and curvature values can be obtained.

Table F.56. Change in forces on deck due to time-dependent effects between t_7 and t_8

$\Delta N_{Dt_7t_8}(kips)$			$\Delta M_{Dt_7t_8}(kips - in.)$		
$\Delta N_{Dt_7t_8}(1)$	$\Delta N_{Dt_7t_8}(2)$	$\Delta N_{Dt_7t_8}(3)$	$\Delta M_{Dt_7t_8}(1)$	$\Delta M_{Dt_7t_8}(2)$	$\Delta M_{Dt_7t_8}(3)$
290	365	391	44	-13	-33

Table F.57. Change in curvature between time t_7 and t_8

$\Delta\phi_t(x) \text{ (1/in.)}$		
$\Delta\phi_t(1)$	$\Delta\phi_t(2)$	$\Delta\phi_t(3)$
5.47E-06	6.92E-06	7.44E-06

Using the previous net curvature values given in Table F.51, and change in curvature values in Table F.57, respectively, the net curvature at final time can be calculated as follows:

Table F.58. Net curvature at final considered time

$\phi_t(8, x) \text{ (1/in.)}$		
$\phi_t(4,1)$	$\phi_t(4,2)$	$\phi_t(4,3)$
5.4E-06	6.77E-06	7.76E-06

Using net curvature values provided in Table F.58 and Eqns. between F-24 and F-31, the net camber at considered final time which is 75 years in this example can be estimated as follows:

Table F.59. Camber at final considered time

Camber (in.)	<i>section index (x)</i>		
	<u>1</u>	<u>2</u>	<u>3</u>
$\Delta_{ns2}(x)$	0	1.91	3.82

F.14. Nomenclature

A_G = girder area, ($in.^2$),
 A_{nm} = area under curvature diagram between n^{th} section and midspan, ($in.^2$)
 A_n = area under curvature diagram between n^{th} section and closest girder end, ($in.^2$)
 A_{ctr} = transformed area of composite structure, ($in.^2$),
 A_{c1tr} = transformed area of 1st composite structure, ($in.^2$),
 A_{c2tr} = transformed area of 2nd composite structure, ($in.^2$),
 A = net area of the element that is going to be transformed, ($in.^2$),
 A_d = area of the deck, ($in.^2$),
 A_{dtr} = transformed area for the deck, ($in.^2$),
 A_{d1tr} = transformed area for the 1st deck, ($in.^2$),
 A_{d2tr} = transformed area for the 2nd deck, ($in.^2$),
 Co_{NG} = coefficient related to axial force on the deck,
 Co_{MG} = coefficient related to moment on the deck,
 E_{CD} = the modulus of elasticity function for deck as a function of time, (ksi),
 E_{CD1} = the modulus of elasticity function for the first deck as a function of time, (ksi),
 E_{CD2} = the modulus of elasticity function for the second deck as a function of time, (ksi),
 E'_{c28} = 28-day modulus of elasticity of concrete, (ksi),
 E_G = modulus of elasticity of girder concrete, (ksi),
 E_D = modulus of elasticity of deck concrete at a specific time, (ksi),
 E_{tr} = modulus of elasticity of the transformed material, (ksi),
 f'_{cdi} = initial concrete compressive strength for deck, (ksi),
 f'_{cd} = 28-day concrete compressive strength for deck, (ksi),
 h = section index array,
 H_r = the average annual ambient relative humidity (%),

i = index for initial reference time,
 I_{dtr} = transformed moment of inertia of the deck, ($in.^4$),
 I_{d1tr} = transformed moment of inertia of the 1st deck, ($in.^4$),
 I_{d2tr} = transformed moment of inertia of the 2nd deck, ($in.^4$),
 I_c = transformed moment of inertia of composite structure, ($in.^4$),
 I_{c1} = transformed moment of inertia of composite structure with 1st deck, ($in.^4$),
 I_{c2} = transformed moment of inertia of composite structure with 2nd deck, ($in.^4$),
 j = section index array,
 k = time index array,
 k_s = factor for the effect of the volume-to-surface ratio of the component
 k_{hc} = humidity factor for creep,
 k_f = factor for the effect of concrete strength,
 k_{tdc} = time development factor for creep,
 k_{tds} = time development factor for shrinkage,
 k_{hs} = humidity factor for shrinkage,
 K_1 = coefficient depending on aggregate type: for normal weight concrete $K_1 = 1.0$,
 m = number of sections to be investigated,
 M_{Gsw} = moment due to girder self-weight, ($kips - in.$),
 M_{co} = matrix composed of coefficients for time-dependent calculations,
 M_c = matrix of constants for time-dependent calculations,
 M_s = solution matrix for time-dependent calculations,
 N_D = array of axial force on the deck a function of time and location, ($kips$),
 M_D = array of moment on the deck a function of time and location, ($kips - in.$),
 M_{d1sw} = moment caused by 1st deck load as a function of location, ($kips - in.$),
 M_{d2sw} = moment caused by 2nd deck load as a function of location, ($kips - in.$),
 M_{b1sw} = moment caused by 1st barrier load as a function of location, ($kips - in.$),

M_{b2sw} = moment caused by 2nd barrier load as a function of location, (*kips – in.*),
 M_{o1sw} = moment caused by 1st overlay load as a function of location, (*kips – in.*),
 M_{o2sw} = moment caused by 2nd overlay load as a function of location, (*kips – in.*),
 M_{d1br} = net moment on the 1st deck before it is removed, (*kips – in.*),
 p = index for final reference time,
 Q_n = first moment of the area under curvature diagram between n^{th} section and closest girder end, (*in.*³),
 r = helper symbol that is related to time reference for time-dependent calculations,
 s_d = average center-to-center spacing of the girders, (*in.*),
 S_{d1} = perimeter of the first deck, (*in.*),
 t_i = age of concrete at the time of load application (*days*),
 t = maturity of concrete, defined as age of concrete between time of loading for creep calculations, or end of curing for shrinkage calculations, and time being considered for analysis of creep or shrinkage effects, (*days*),
 t_d = average deck thickness, (*in.*),
 t_h = average haunch thickness, (*in.*),
 t_{o1} = 1st overlay thickness, (*in.*),
 t_{o2} = 2nd overlay thickness, (*in.*),
 u = time index array,
 V_{d1} = volume of the first deck, (*in.*³),
 V/S = volume-to-surface ratio (*in.*),
 w = unit weight of concrete, (*kcf*),
 w_G = load due to girder self-weight, *kips/in.*,
 w_{d1} = load due to 1st deck, (*kips/in.*),
 w_{d2} = load due to 2nd deck, (*kips/in.*),
 w_{b1} = load due to 1st barrier load, (*kips/in.*),
 w_{b2} = load due to 2nd barrier load, (*kips/in.*),
 w_{o1} = load due to 1st overlay load, (*kips/in.*),
 w_{o2} = load due to 2nd overlay load, (*kips/in.*),
 x_n = location of n^{th} section measured from closest girder end, (*in.*),

X_{Gof} = fabricated curvature along the span, (*1/in.*),
 X_n = curvature at n^{th} section, (*1/in.*),
 x_m = location of midspan measured from girder end, (*in.*),
 X_m = curvature at midspan, (*1/in.*),
 $\bar{x}_n$ = centroid of the area under curvature diagram between n^{th} section and closest girder end, (*in.*),
 y_{dG} = a parameter indicating the distance between centroids of girder and deck, (*in.*),
 $y_{Gbottom}$ = centroid of girder with respect to bottom of girder, (*in.*),
 y_{dc} = centroid of the deck & haunch couple with respect to the bottom of the haunch, (*in.*),
 $y_{dbottom}$ = distance between centroid of deck & haunch couple with respect to bottom of the girder, (*in.*),
 y_c = distance of centroid of transformed composite structure to the bottom of girder, (*in.*),
 y_{c1} = distance of centroid of transformed composite structure to the bottom of girder with 1st deck, (*in.*),
 y_{c2} = distance of centroid of transformed composite structure to the bottom of girder with 2nd deck, (*in.*),
 z = time index array,
 loc = array for location, (*in.*),
 $stopper$ = helper word for time-dependent calculations,
 $continue$ = loop term that restarts the current iteration,
 α = parameters for ACI209R-92 development model depending on curing type,
 α_1 = coefficient used to determine fabricated camber along the span,
 α_2 = coefficient used to determine fabricated curvature along the span,
 β = parameters for ACI209R-92 development model depending on cement type,
 $\theta_{B/C}$ = relative slope at point B relative to point B, (*rad.*),

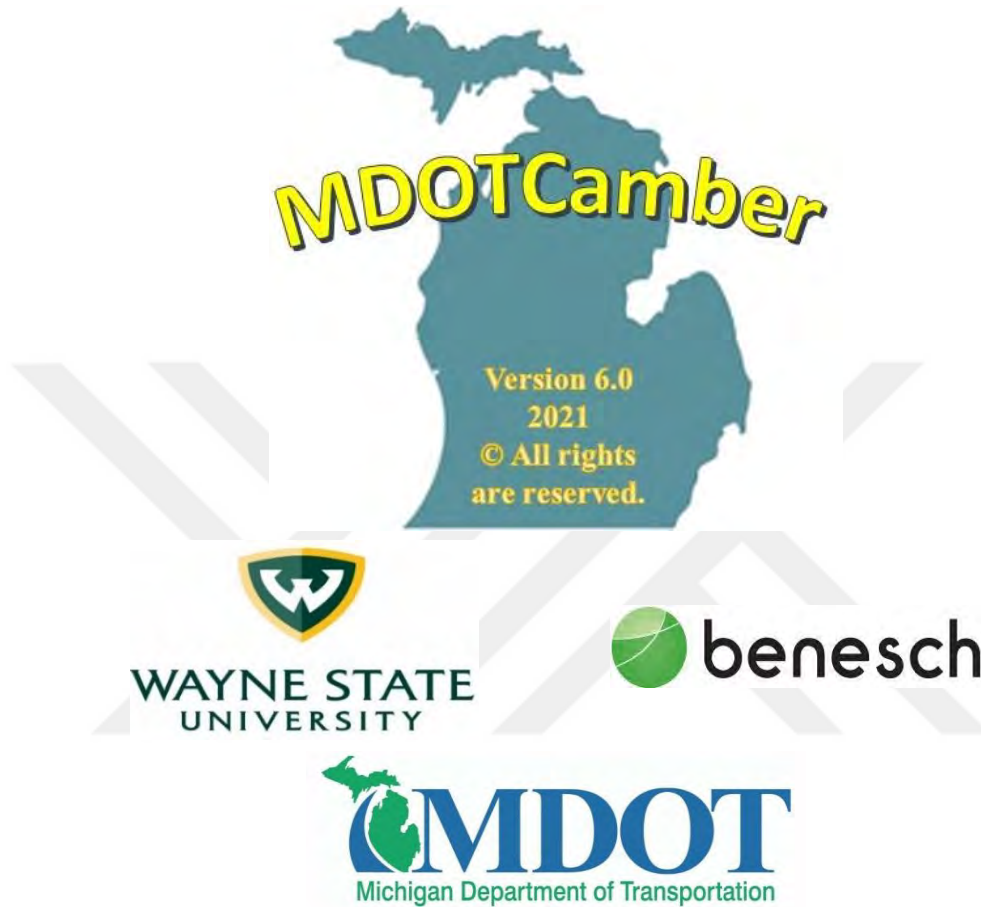
γ_G = unit-weight of girder, (*kcf*),
 γ_{o1} = unit-weight of 1st overlay, (*pcf*),
 γ_{o2} = unit-weight of 2nd overlay, (*pcf*),
 ε_{shD} = remaining deck shrinkage at a given time, (*in./in.*),
 β_{ax} = magnification factor tension creep,
 β_{flex} = magnification factor flexural creep,
 ψ_{crD} = remaining deck creep at a given time,
 $\psi_{crD,flex}$ = remaining deck flexural creep at a given time,
 $\psi_{crG,ax}$ = remaining girder tension creep at a given time,
 $\psi_{crD,ax}$ = remaining deck tension creep at a given time,
 $\psi(t, t_i)$ = the creep coefficient at concrete at t due to load applied at t_i ,
 ϕ_{pre} = net curvature after girder erection, (*1/in.*),
 ϕ_0 = initial curvature array as a function of time and location, (*1/in.*),
 ϕ_p = initial curvature array as a function of time and location, (*1/in.*),
 $\Delta\phi_t$ = change in curvature due to time-dependent effects as a function of time and location, (*1/in.*),
 ϕ_{ad1p} = net curvature just after 1st deck is placed, (*1/in.*),
 ϕ_{ad2p} = net curvature just after 2nd deck is placed, (*1/in.*),
 ϕ_{naas1} = net curvature just after application of barrier and overlay loads on 1st superstructure, (*1/in.*),
 ϕ_{nbndp} = net curvature before new deck placement, (*1/in.*),
 ϕ_{naas2} = net curvature just after application of barrier and overlay loads on 2nd superstructure, (*1/in.*),
 ϕ_{ad1r} = net curvature just after deck removal, (*1/in.*),
 η = modular ratio,
 η_D = modular ratio for the deck
 η_{D1} = modular ratio for the 1st deck,
 η_{D2} = modular ratio for the 2nd deck,
 Δ_{Gof} = fabricated camber along the span, (*in.*),

Δ_{pre} = pre-erection camber along the span, (*in.*),
 Δ_{nad1p} = camber just after placement of 1st deck, (*in.*),
 Δ_{nbas1} = camber just before application of barrier and overlay loads on 1st superstructure, (*in.*),
 Δ_{nbas2} = camber just before application of barrier and overlay loads on 2nd superstructure, (*in.*),
 Δ_{naas1} = camber just after application of barrier and overlay loads on 1st superstructure, (*in.*),
 Δ_{naas2} = camber just after application of barrier and overlay loads on 2nd superstructure, (*in.*),
 Δ_{nbd1r} = camber just before deck removal process, (*in.*),
 Δ_{nad1r} = camber just after deck removal process, (*in.*),
 Δ_{nbndp} = camber just before new deck placement, (*in.*),
 Δ_{nad2p} = camber just after placement of 2nd deck, (*in.*),
 Δ_{ns2} = camber at final considered time, (*in.*),
 Δ_n = net deflection at n^{th} section according to end of the girder, (*in.*),
 $\Delta_{A/C}$ = relative displacement at point C according to point A, (*in.*),
 Δx = length of a section, (*in.*),
 ΔM_{d1ob1r} = change in moment on the 1st deck due to removal of barrier and overlay loads, (*kips – in.*),
 ΔM_{Gd1swr} = change in moment on girder due to deck self-weight removal, (*kips – in.*),
 ΔM_{d1b1sw} = change in moment on the 1st deck due to 1st barrier load, (*kips – in.*),
 ΔM_{Gb2sw} = change in moment on the girder due to 2nd barrier load, (*kips – in.*),
 ΔM_{d2b2sw} = change in moment on the 2nd deck due to 2nd barrier load, (*kips – in.*),
 ΔM_{d1o1sw} = change in moment on the 1st deck due to 1st overlay load, (*kips – in.*),

ΔM_{d2o2sw} = change in moment on the 2nd deck due to 2nd overlay load, (*kips – in.*),
 $\Delta M_{Dt_3t_4}$ = change in moment on deck due to time-dependent effects between t_3 and t_4 , (*kips – in.*),
 $\Delta M_{Dt_7t_8}$ = change in moment on deck due to time-dependent effects between t_7 and t_8 , (*kips – in.*),
 ΔN_{d1ob1r} = change in axial force on the 1st deck due to removal of barrier and overlay loads, (*kips*),
 ΔN_{d1b1sw} = change in axial force on the 1st deck due to 1st barrier load, (*kips*),
 ΔN_{d2b2sw} = change in axial force on the 2nd deck due to 2nd barrier load, (*kips*),
 ΔN_{d1o1sw} = change in axial force on the 1st deck due to 1st overlay load, (*kips*),
 ΔN_{d2o2sw} = change in axial force on the 2nd deck due to 2nd overlay load, (*kips*),
 $\Delta N_{Dt_3t_4}$ = change in axial force on deck due to time-dependent effects between t_3 and t_4 , (*kips – in.*),
 $\Delta N_{Dt_7t_8}$ = change in axial force on deck due to time-dependent effects between t_7 and t_8 , (*kips – in.*),
 $\Delta \emptyset_{sc}$ = change in curvature due to change in support conditions, (*1/in.*),
 $\Delta \phi_{Gsw}$ = change in curvature due to girder self-weight, (*1/in.*),
 $\Delta \emptyset_{d1sw}$ = change in curvature due to 1st deck self-weight, (*1/in.*),
 $\Delta \emptyset_{d2sw}$ = change in curvature due to 2nd deck self-weight, (*1/in.*),
 $\Delta \emptyset_{b1sw}$ = change in curvature due to 1st barrier self-weight, (*1/in.*),
 $\Delta \emptyset_{b2sw}$ = change in curvature due to 2nd barrier self-weight, (*1/in.*),
 $\Delta \emptyset_{o1sw}$ = change in curvature due to 1st barrier self-weight, (*1/in.*),
 $\Delta \emptyset_{o2sw}$ = change in curvature due to 2nd barrier self-weight, (*1/in.*),
 $\Delta \emptyset_{ob1r}$ = change in curvature due to removal of barrier and overlay loads, (*1/in.*),

$\Delta \emptyset_{d1r}$ = change in curvature due to removal of locked forces inside the deck, (*1/in.*),
 $\Delta \emptyset_{d1swr}$ = change in curvature due to deck self-weight removal, (*1/in.*),

**APPENDIX G: Example Problem for Predicting Camber and Deflection for S11 Project
using *MDOTCamber* Program**



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May 2021

G.1. Problem Statement

Using *MDOTCamber* v6.0 program, determine camber and deflection history by including temperature gradient effects for the fascia precast bulb tee beam used in the I-75 Modernization S11 Bridge project from casting of girder to the end of service life, which is given as 75 years. In addition to camber and deflection history, calculate the screed cambers along the span to determine the slab and screed elevations.

The beam used in S11 project is a type of bulb tee beam with 49" top flange width and 54" height. The design length of the beam is 136'-9", and it is supported $L_t = 21.5"$ and $L_b = 5.5"$ away from girder ends at precast facility and bridge site, respectively (Figure G.1).

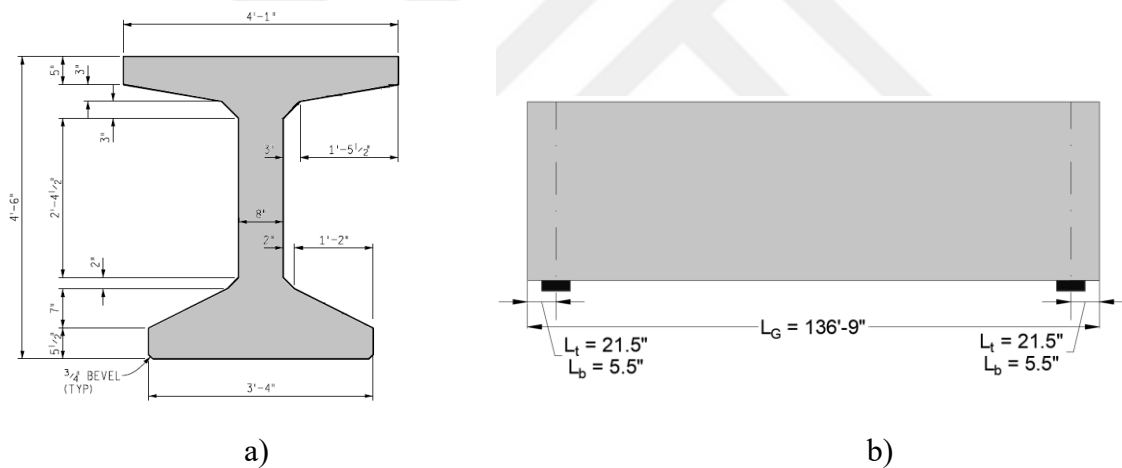


Figure G.1. a) Cross-sectional dimensions for 49" x 54" deep bulb tee and b) beam length (L_G) and support conditions at precast facility (L_t), and bridge site (L_b)

Beam concrete is a normal weight concrete and, specified concrete compressive strengths at prestress transfer and 28 days are $f'_{ci} = 7$ ksi and $f'_c = 10$ ksi, respectively. Use AASHTO LRFD (2020) Table 3.5.1-1 when calculating unit weight of plain concrete. Assume 5 pcf mild and prestressing reinforcement weight when considering self-weight calculations. Center-to-center spacing between the girders is 8'-5" and distance between the edge of deck and Φ of the fascia beam is 3'-2 3/4" (Figure G.2).

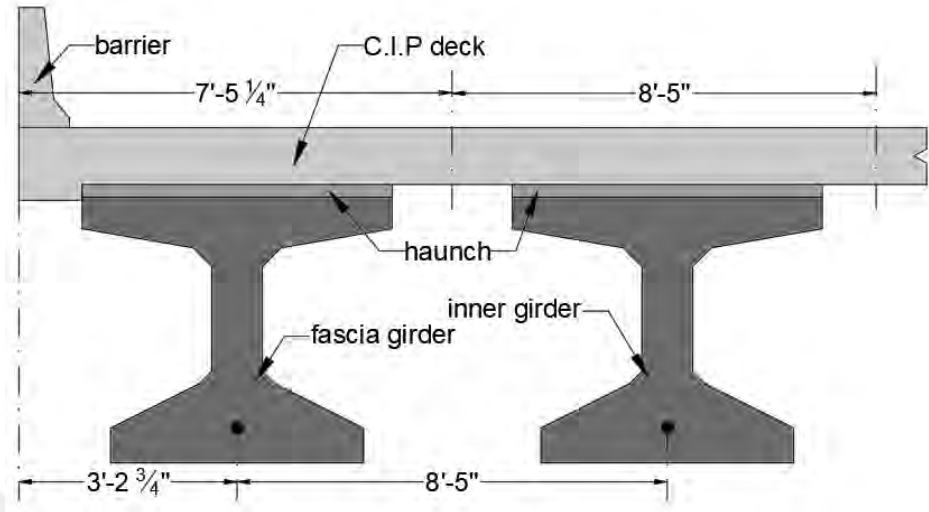


Figure G.2. Spacing for inner and fascia girders

For the temperature gradient analysis, use zone III, and assume coefficient of thermal expansion for normal weight concrete as $6.0 \cdot 10^{-6} \text{ } 1/^{\circ}\text{F}$ according to AASHTO LRFD (2020) section 5.4.2.2. Use AASHTO LRFD (2020) model for the estimation of modulus of elasticity of girder concrete and ACI 209R-92 model with back-calculated parameters for its variation with time. Assume aggregate factor, K_1 , equals to 1.0. Use Michigan overstrength factors to account for actual compressive strength of girder concrete.

Concrete used in decks is a normal weight concrete with 4 ksi specified 28 days strength: $f'_{cd} = 4 \text{ ksi}$. Deck thickness is given as 9" and, 3" additional thickness is provided for overhang portion as illustrated in Figure G.3. Use the average deck thickness when considering deck thickness for the fascia girder. Assume the average haunch thickness is 2 in. Consider 2 in. of overlay load with unit weight of 140 pcf and Type IV barrier in accordance with MDOT Bridge Design Guides section 6.29.09 (Figure G.3).

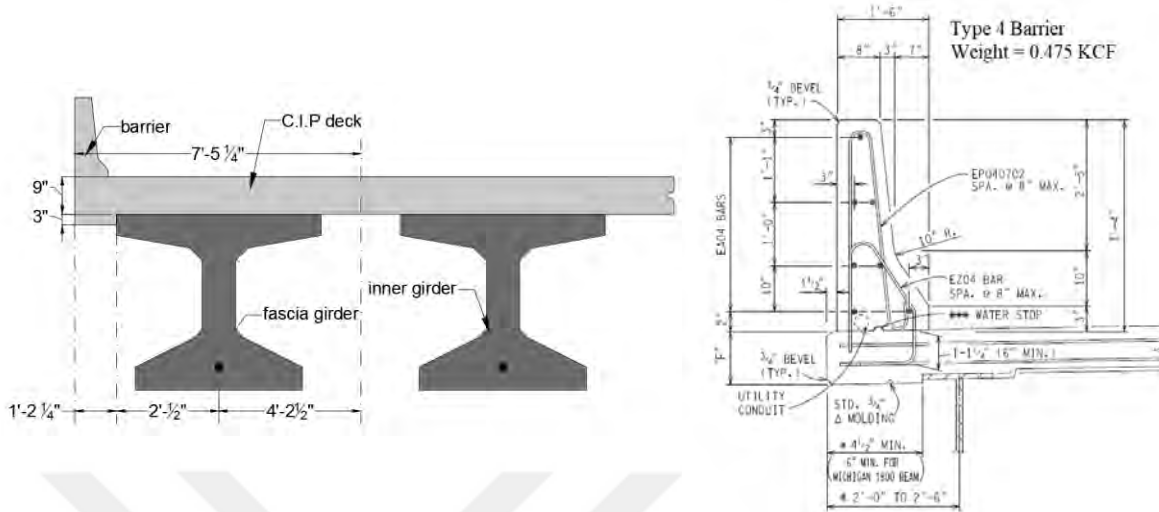


Figure G.3. Detailed dimensions for fascia girder and barrier

For screed camber calculation, use the following loads in accordance with Michigan Design Manual Bridge Design Chapter 7 LRFD section 7.02.22:

- 10 psf formwork load,
- 10 psf rebar load,
- 0.5 kips diaphragm load (consider only midspan diaphragm: $N = 1$) (Figure G.4)

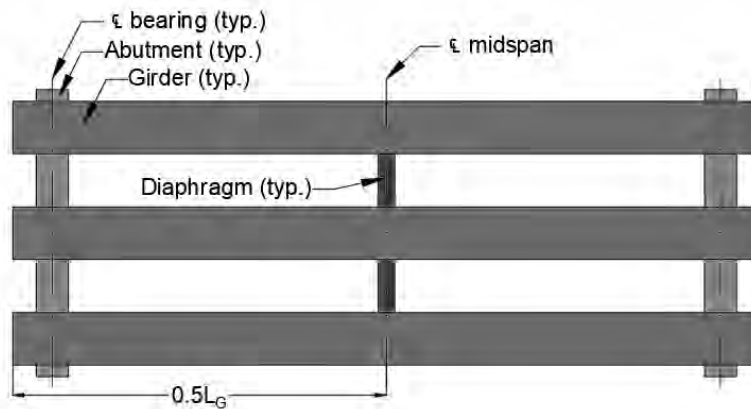


Figure G.4. Diaphragm location

For the prestressing steel, $\emptyset 0.6$ " diameter low-relaxation strands are used with ultimate tensile strength, $f_{pu} = 270$ ksi. Modulus of elasticity for prestressing strands is given as $E_{ps} =$

28500 ksi and, jacking and yield stress for strands are given as $f_{pj} = 0.75f_{pu}$ and $f_{py} = 0.9f_{pu}$, respectively. The strand layout at midspan and end locations as well as debonded strand information are provided in Figure G.5.

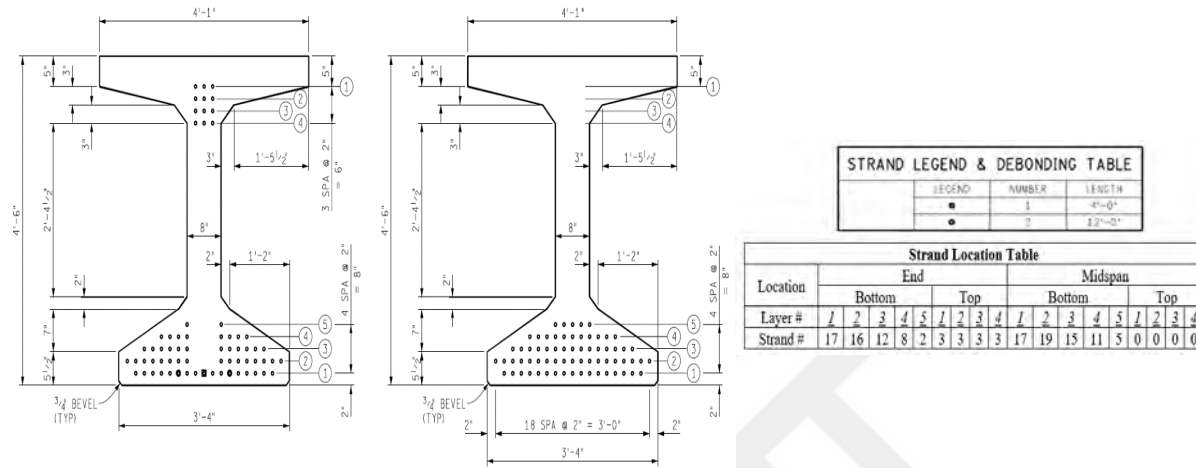


Figure G.5. Strand layout and debonding strand information

Harped strands are harped at $0.33L_G$ away from girder ends, where L_G is the girder length.

For creep and shrinkage models, use AASHTO LRFD (2020) Body provision. Assume tensile and flexural creep factors equal to 1.0. Use 51 sections in the analysis.

It is given that girder is steam cured for 1 day, and the decks are moist cured for 7 days. Average ambient relative humidity, H , is given as 70%. For the construction timeline, use the values provided in Table G.1. Use “proposed method” for the generation of time matrix.

Table G.1. Construction timeline

Event	Time (days)
Detensioning of strands (t_1)	1
Time at which the deck is placed (t_2)	28
Time at placement of barriers and the opening of bridge to traffic (t_3)	35
Time at placement of overlay (t_4)	35
Time at deck removal (t_5)	18250
Time at new deck placement (t_6)	18260
Time at application of barrier load and reopening of bridge to traffic (t_7)	18267
Time at placement of overlay (t_8)	18267
Time at the end of service life (t_9)	27375

G.2. Solution Steps

Open *MDOTCamber* v6.0 program. There are 8 tabs in the program where six of them are for the project inputs, one is for the general project and analysis information and one is for the results. For the solution procedure, inputs under the following tabs are provided in given order:

- 1) Tab 1: Introduction (General project and analysis information)
- 2) Tab 2: Girder Properties
- 3) Tab 3: Girder Concrete Properties
- 4) Tab 4: Deck Properties
- 5) Tab 5: Prestressing Steel Properties
- 6) Tab 6: Creep and Shrinkage Model Selection
- 7) Tab 7: Time, Environment and Measured Camber Properties

G.2.1. Tab 1: Introduction

Under the “Introduction” tab, provide general information about the project; name of project, job number, analysis conducted by, analysis checked by, and date (Figure G.6). The software allows you to upload project data if you have conducted the analysis in the past and wish to redo the analysis to change a parameter or retrieve previous results. The “i” button next to

“About” provides general information about the software as well as software limitations. The sign convention used in the analysis is presented at the bottom.

The screenshot displays the MDOTCamber v6.0 software window. The title bar reads "MDOTCamber v6.0". The menu bar includes "Introduction", "Girder Properties", "Girder Concrete Properties", "Deck Properties", and "Prestressing Steel I". The main content area is titled "Predicting Camber and Deflections in Composite Bridge Superstructures". It features logos for MDOT (Michigan Department of Transportation), Wayne State College of Engineering, and benesch. Below the logos, there are input fields for project information: "Name of the Project:" (S11), "Job Number:" (201437), "Conducted By:" (Furkan Cakmak), "Checked By:" (Fatmir Menkulasi), "Date:" (Mar 05, 2021), and "Project Data:" (Load Project Data). A "Sign Convention" section is located at the bottom, showing diagrams for: (-) Upward Deflection, (+) Downward Deflection, (-) Negative Curvature, (+) Positive Curvature, (-) Negative Moment, (+) Positive Moment, (-) Compression, and (+) Tension. The copyright notice at the bottom right states "Copyright © 2021 MDOTCamber. All rights reserved."

Figure G.6. General project and analysis information

G.2.2. Tab 2: Girder Properties

Before providing inputs for the girder, firstly, beam and analysis type should be selected. After the selections are made, they are locked by clicking the button shown below to be able to continue the analysis (Figure G.7).

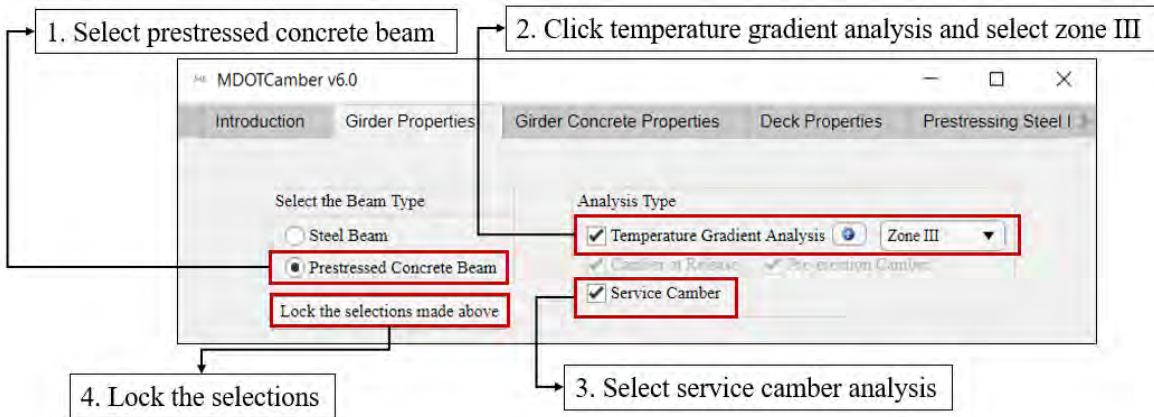


Figure G.7. Steps to be followed before the selection of girder type

After locking the selections, a dropdown appears in the program. Select the Bulb Tee Beam with 49" Top Flange Width and 54" Height as highlighted in the red box since this is the beam type used in S11 project. The program then calculates beam section properties automatically.

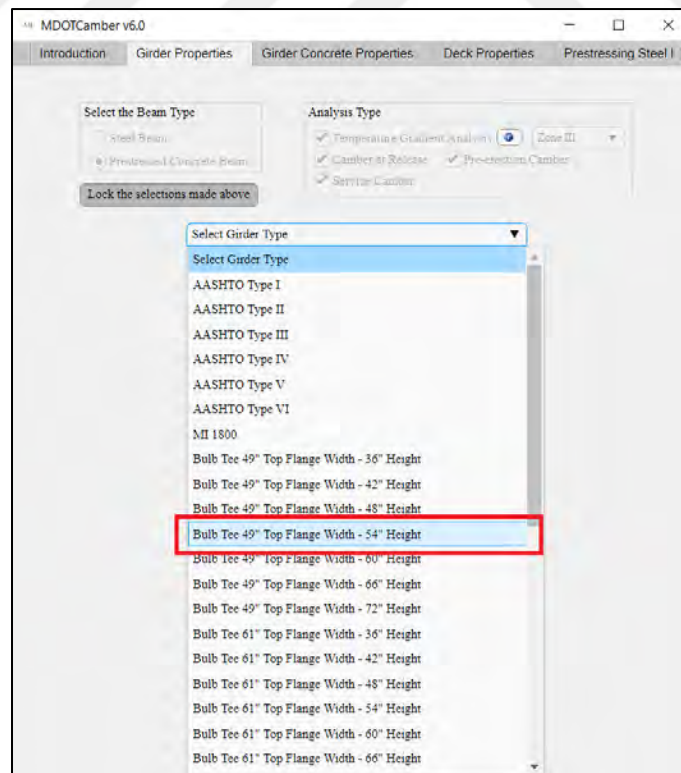


Figure G.8. Selection of girder type

After the girder type is selected, a table appears in the program which requires additional information about the beam. Provide the required information by using the provided inputs, fill the table as shown in Figure G.9.

1. Unit weight of plain concrete plus the weight of mild and prestressing steel should be considered. According to AASHTO LRFD (2020) Table 3.5.1-1, for the concrete whose 28 days strength is more than 5 ksi, the unit weight of plain concrete:

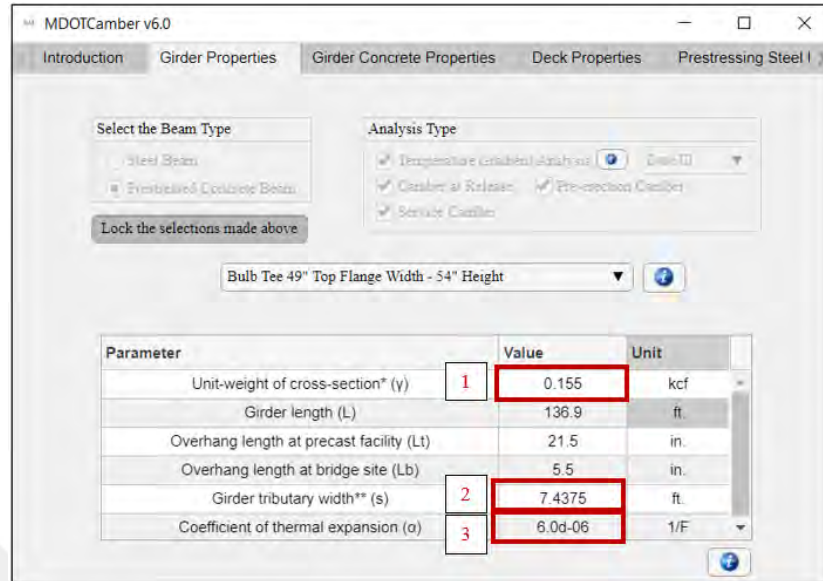
$$\gamma_c = 0.140 + 0.001 \cdot f'_c = 0.140 + 0.001 \cdot 10 = 0.150 \text{ kcf}$$

By adding 5 pcf for weight of mild and prestressing steel, the unit weight of cross-section is calculated as 0.155 kcf.

2. For the fascia beam, the girder tributary width is calculated by taking half of the center-to-center girder spacing and distance between the edge of deck and Φ of the fascia beam (Figure G.2):

$$s = \frac{8 + 5/12}{2} + \left(3 + \frac{2.75}{12}\right) = 7.4375 \text{ ft.}$$

3. Based on AASHTO LRFD 2020 section 5.4.2.2, the coefficient of thermal expansion for normal weight concrete is $6.0 \cdot 10^{-6} \text{ } 1/^{\circ}\text{F}$.



MDOTCamber v6.0

Introduction Girder Properties Girder Concrete Properties Deck Properties Prestressing Steel I >

Select the Beam Type

☐ Steel Beam
☒ Prestressed Concrete Beam

Lock the selections made above

Analysis Type

☒ Temperature Gradient Analysis ☐ Design II
☒ Girders at Release ☒ Pre-tension Camber
☒ Service Camber

Bulb Tee 49" Top Flange Width - 54" Height

Parameter		Value	Unit
Unit-weight of cross-section* (γ)	1	0.155	kcf
Girder length (L)		136.9	ft.
Overhang length at precast facility (Lt)		21.5	in.
Overhang length at bridge site (Lb)		5.5	in.
Girder tributary width** (s)	2	7.4375	ft.
Coefficient of thermal expansion (α)	3	6.0d-06	1/F

Figure G.9. Additional information about girder properties

G.2.3. Tab 3: Girder Concrete Properties

After providing all the inputs for girder properties in Tab 2, now proceed for the next tab which is “Girder Concrete Properties”, and do the followings (Figure G.10):

1. Select “ACI 209R-92” model for variation of Modulus of Elasticity with time.
2. Provide specified concrete compressive strength values at prestress transfer, f'_{ci} , and 28 days, f'_c , as 7 ksi and 10 ksi, respectively.
3. Select “Michigan Factors” as an overstrength factor to account for actual compressive strengths.
4. Select “AASHTO LRFD 2020” model for estimating modulus of elasticity values at prestress transfer and 28 days.
5. Select “Back-calculate parameter” option for the parameters of ACI 209R-92 model.
6. According to AASHTO LRFD (2020) Table 3.5.1-1, for the concrete whose 28 days strength is more than 5 ksi, the unit weight of plain concrete:

$$\gamma_c = 0.140 + 0.001 \cdot f'_c = 0.140 + 0.001 \cdot 10 = 0.150 \text{ kcf}$$

- Enter 1.0 for “Aggregate Factor, (K_1)”.

MDOTCamber v6.0

Introduction | Girder Properties | **Girder Concrete Properties** | Deck Properties | Prestressing Steel

A. Variation of Modulus of Elasticity with Time

1. ACI 209R-92 (Recommended)

B. Concrete Compressive Strength at Release and at 28 Days

☐ Click to provide actual concrete compressive strength values.

Specified Concrete Compressive Strength

Symbol	Definition	Value (ksi)
f_{ci}	Specified concrete compressive strength at release	7
f'_c	Specified concrete compressive strength at 28 day	10

2.

Overstrength Factors for Concrete Strength

3. Michigan Factors (Recommended)

Symbol	Definition	Factor
OSi	Overstrength factor for concrete at release	1.2
OS28	Overstrength factor for concrete at 28 day	1.2

C. Modulus of Elasticity at Release and at 28 Days

☐ Click to provide actual modulus of elasticity values.

Model for estimating elastic modulus: AASHTO LRFD 2020 (Recommended)

4.

Required Parameters for modulus of elasticity

5. ☒ Back-calculate parameters.

6. Unit-weight of plain concrete (wc): 0.15 kcf

7. Aggregate factor (K_1): 1

Figure G.10. Inputs for the girder concrete properties

G.2.4. Tab 4: Deck Properties

After providing all necessary inputs for the girder concrete properties, move on the “Deck Properties” tab to provide inputs for the screed computations, and 1st and 2nd decks, and do the followings (Figure G.11):

- Provide parameters for calculating slab and screed elevations as given in the problem statement (Figure G.4):
 - 10 psf for formwork;
 - 10 psf for rebars;

- 0.5 kips for diaphragm;
 - Enter 1.0 value for considering only diaphragms at midspan (Figure G.4).
2. Click “Yes” to consider deck replacement event.
 3. Provide parameters for the 1st and 2nd decks:
 - i. Deck concrete compressive strength is given as 4 ksi.
 - ii. According to AASHTO LRFD (2020) Table 3.5.1-1, for concretes whose 28 days strength is less than 5 ksi, the unit weight of plain concrete is: $\gamma_c = 145$ pcf. By adding 5 pcf for the rebars, the unit weight for the reinforced deck becomes: $\gamma_d = 150$ pcf = 0.150 kcf.
 - iii. The average deck thickness for the fascia girder can be calculated as follows (Figure G.3):

$$t_d = \frac{9 \cdot [(2 \cdot 12 + 0.5) + (4 \cdot 12 + 2.5)] + (3 + 9) \cdot (1 \cdot 12 + 2.25)}{(1 \cdot 12 + 2.25 + 2 \cdot 12 + 0.5 + 4 \cdot 12 + 2.5)} = 9.48 \text{ in.}$$
 - iv. Aggregate factor for deck concrete is given as 1.0.
 - v. Haunch thickness is given as 2 in.
 - vi. Overlay thickness and unit weight are given as 2 in. and 0.140 kcf, respectively.
 - vii. The weight for Type IV barrier is provided as 0.475 klf (Figure G.3)
 - viii. Parameters for the first and second decks are considered to be identical.

1

Parameter	Value	Unit
Slab formwork load (wsf)	10	psf
Surface load due to rebar (wsr)	10	psf
Diaphragm load (Pd)	0.5	kips
Number of interior diaphragms* (Ndp)	1	-

2

Has the deck been replaced?

☒ Yes

☐ No

3

Parameter for the First Deck	Value	Unit
Specified concrete compressive strength at 28 day (fcd)	4	ksi
Unit-weight of deck and haunch* (with mild steel) (γ)	0.150	kcf
Average deck thickness** (td)	9.48	in.
Aggregate factor ¹ (K1)	1	-
Unit weight of plain concrete ¹ (wc)	0.145	kcf
Average haunch thickness (th)	2	in.
Overlay thickness (to)	2	in.
Unit-weight of overlay (γ_o)	0.140	kcf
Barrier load ^{1/2} (wb)	0.475	klf

Parameter for the Second Deck	Value	Unit
Specified concrete compressive strength at 28 day (fcd)	4	ksi
Unit-weight of deck and haunch* (with mild steel) (γ)	0.150	kcf
Average deck thickness** (td)	9.48	in.
Aggregate factor ¹ (K1)	1	-
Unit weight of plain concrete ¹ (wc)	0.145	kcf
Average haunch thickness (th)	2	in.
Overlay thickness (to)	2	in.
Unit-weight of overlay (γ_o)	0.140	kcf
Barrier load ^{1/2} (wb)	0.475	klf

Figure G.11. Inputs for deck properties

G.2.5. Tab 5: Prestressing Steel Properties

After providing all the inputs for the deck properties, proceed for the next tab: “Prestressing Steel Properties”, and do the followings (Figure G.12):

1. Check this box because there are harped strands.
2. Provide general prestressing strand properties:

- i. $E_{ps} = 28500$ ksi (AASHTO LRFD 2020 section 5.4.4.2)
 - ii. $d_{ps} = 0.6$ in. and $f_{pu} = 270$ ksi
 - iii. $f_{pj} = 0.75 \cdot f_{pu} = 0.75 \cdot 270 = 202.5$ ksi (AASHTO LRFD 2020 Table 5.9.2.2-1)
 - iv. $f_{py} = 0.90 \cdot f_{pu} = 0.90 \cdot 270 = 243$ ksi (AASHTO LRFD 2020 Table 5.4.4.1-1)
 - v. Strand type = Low-relaxation $\rightarrow$ Enter 1.0 for this.
3. Using "+" and "-" button, provide strand layout information as provided in Figure G.5.
- 1st table is for the fully bonded straight and debonded strands, and 2nd table is for the harped strands.
4. Debonding length for fully bonded strands must be 0.
5. Strands are harped at $0.33L_G = 0.33 \cdot \left(136 + \frac{9}{12}\right) = 45.1275$ ft. away from girder ends.

MDOTCamber v6.0

Girder Properties Girder Concrete Properties Deck Properties Prestressing Steel Properties

☒ Check if there is any harped strand. 1

2

Parameter	Value	Unit
Modulus of elasticity (Eps)	28500	ksi
Nominal diameter (dps)	0.6	in.
Ultimate tensile strength (fpu)	270	ksi
Jacking stress (fpj)	202.5	ksi
Yield stress (fpy)	243	ksi
Strand Type	1	1:Low-r...

Layer Group	Group Type	Number of Strands in Layer Group	Distance from Bottom of Girder (in.)	Debonded Length (ft.)
1	Fully Bonded, Straight	14	2	0
2	Fully Bonded, Straight	16	4	0
3	Fully Bonded, Straight	12	6	0
4	Fully Bonded, Straight	8	8	0
5	Fully Bonded, Straight	2	10	0
6	Debonded, Straight	2	2	12
7	Debonded, Straight	1	2	4

Layer Group	Group Type	Number of Strands in Layer Group	Distance from Bottom of Girder at Midspan (in.)	Distance from Bottom of Girder at End (in.)	Harping Point (ft.)
1	Harped	2	4	45.1275	45.1275
2	Harped	3	6	45	45.1275
3	Harped	3	8	47	45.1275
4	Harped	2	10	49	45.1275

3

4

5

Figure G.12. Prestressing Steel Properties

G.2.6. Tab 6: Creep and Shrinkage Model Selection

Following the strand properties, proceed for the next tab which is “Creep and Shrinkage Model Selection”. As provided in the problem statement, select AASHTO Body 2020 model, and assign creep magnification factors i.e. both flexural and tension to 1.0 (Figure G.13).

Note: After the model is selected, the parameters corresponding to the selected model should be provided. If they are not known, default values (shown below if appropriate) will be assigned to them.

Creep and Shrinkage Model: AASHTO Body 2020 (Recommended)

Parameter	Magnification Factor
Tensile creep magnification factor	1
Flexural creep magnification factor	1

Parameters needed to be defined:
AASHTO Body 2020

For this model, no parameter is required.

Figure G.13. Selection of creep and shrinkage model and assigning creep magnification factors

G.2.7. Tab 7: Time, Environment and Measured Camber Properties

After the selection of creep and shrinkage model, information about the construction timeline and number of sections to be investigated is provided under “Time, Environment and Measured Camber Properties”. The steps in this tab are as follows:

1. Enter “51” value for the number of sections to be investigated.
2. Provide construction timeline based on provided inputs. Average ambient relative humidity value was given as 70 %. Also, provide the number of curing days for the girder and decks as 1, 7 and 7 days, respectively.
3. Select proposed method for the generation of time matrix.

4. Leave this box unchecked as there is not measured camber data provided in the problem statement.

MDOTCamber v6.0

Creep and Shrinkage Model Selection Time, Environment and Measured Camber Properties Results

Number of sections to be investigated: S1 1

A. Time and Environment Properties

Parameter	Value	Unit
Relative humidity* (H)	70	%
Age of girder when prestressing strands are released (t1)	1	days
Age of girder at deck placement (t2)	28	days
Age of girder when barrier load is applied on first super-structure (t3)	35	days
Age of girder when overlay load is applied on first super-structure (t4)	35	days
Age of girder at deck removal (t5)	18250	days
Age of girder at new deck placement (t6)	18260	days
Age of girder when barrier load is applied on second super-structure...	18267	days
Age of girder when overlay load is applied on second super-structure...	18267	days

Time Matrix Generation

3 Proposed (Recommended) Gilbert et al. (2010)

A logarithmic timescale will be assigned for conducting time-dependent calculations. Please consult user's manual for detailed information.

B. Measured Camber Properties

☐ Check if there is any measured camber value. 4

Figure G.14. Inputs for construction timeline and environment properties

G.3. Results - General

After providing all the required inputs to be able to conduct the analysis, proceed for the last tab which is “Results” tab. Click on “calculate” button to initiate the calculations, and wait until the analysis is finished. It takes less than 60 seconds in i5 9300H 2.3 GHz processor with 8 GB of RAM capacity. The explanations for the results presented in this tab as follows:

1. Table shows the camber with and without temperature gradient effect at discrete times which can be downloaded using “Download Data” on right top of the table.

2. Figure shows the camber and displacement history with temperature gradient effects.
3. Table shows the camber and displacement history data, which can be downloaded using the “Download Data” button next to it.
4. Project specific information which was provided in the “Introduction” tab.
5. Options are provided to print and save the project input data.
6. Additional results such as deflected shape of the girder, prestressing strand stress history, and girder and deck stress histories can be reached using these buttons (Figure G.17 and Figure G.18).
7. Information about slab and screed guidance can be obtained using this button (Figure G.16).

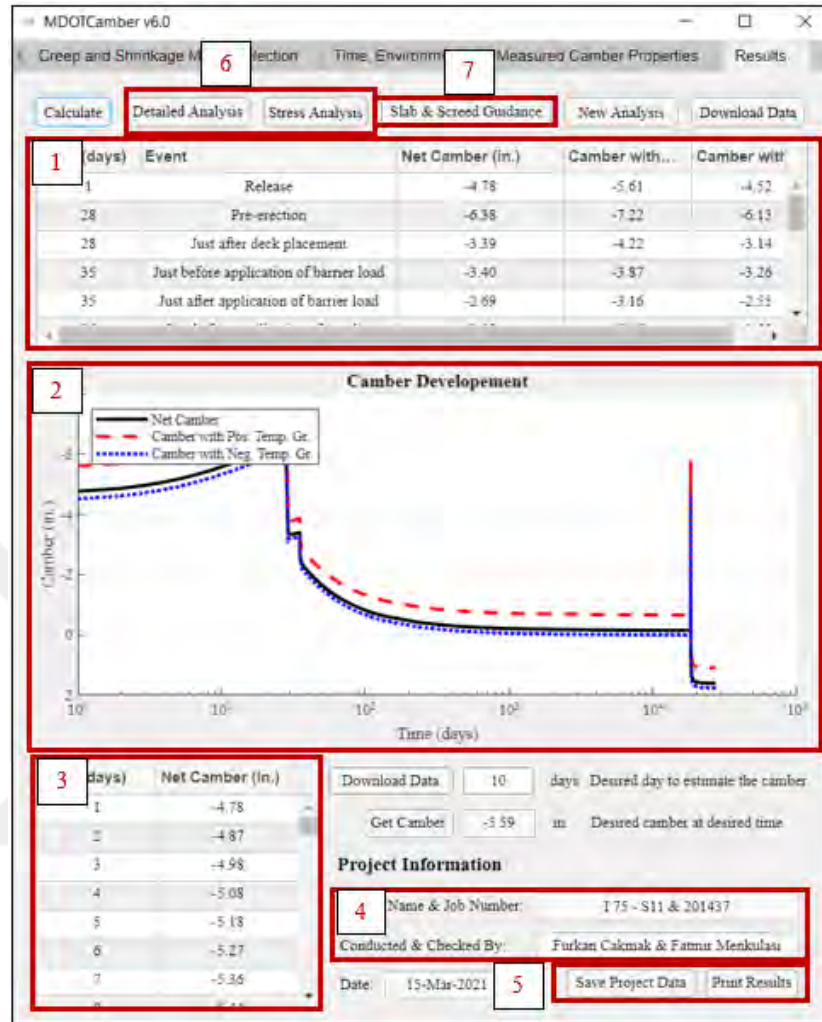


Figure G.15. General results

MDOTCamber v6.0

For the first super-structure

Download Data

Location (ft.)	Net Camber just after girder erection (in.)	Net Camber just after app. of diaphragm load (in.)	Net Camber just after app. of formwork and deck rein. (in.)	Net Camber just after app. of deck load (in.)
0	0	0	0	0
1.37	-0.25	-0.25	-0.24	-0.16
2.74	-0.51	-0.51	-0.48	-0.31
4.11	-0.75	-0.74	-0.70	-0.45
5.48	-1.00	-1.00	-0.94	-0.61
6.84	-1.22	-1.21	-1.14	-0.73
8.21	-1.46	-1.45	-1.30	-0.88

For the second super-structure

Download Data

Location (ft.)	Net Camber of girder alone on bridge site (in.)	Net Camber just after app. of diaphragm load (in.)	Net Camber just after app. of formwork and deck rein. (in.)	Net Camber just after app. of deck load (in.)
0	0	0	0	0
1.37	-0.22	-0.22	-0.21	-0.13
2.74	-0.45	-0.44	-0.42	-0.25
4.11	-0.65	-0.65	-0.61	-0.36
5.48	-0.87	-0.87	-0.81	-0.48
6.84	-1.05	-1.05	-0.98	-0.57
8.21	-1.17	-1.16	-1.10	-0.68

Figure G.16. Data for obtaining slab and screed elevations

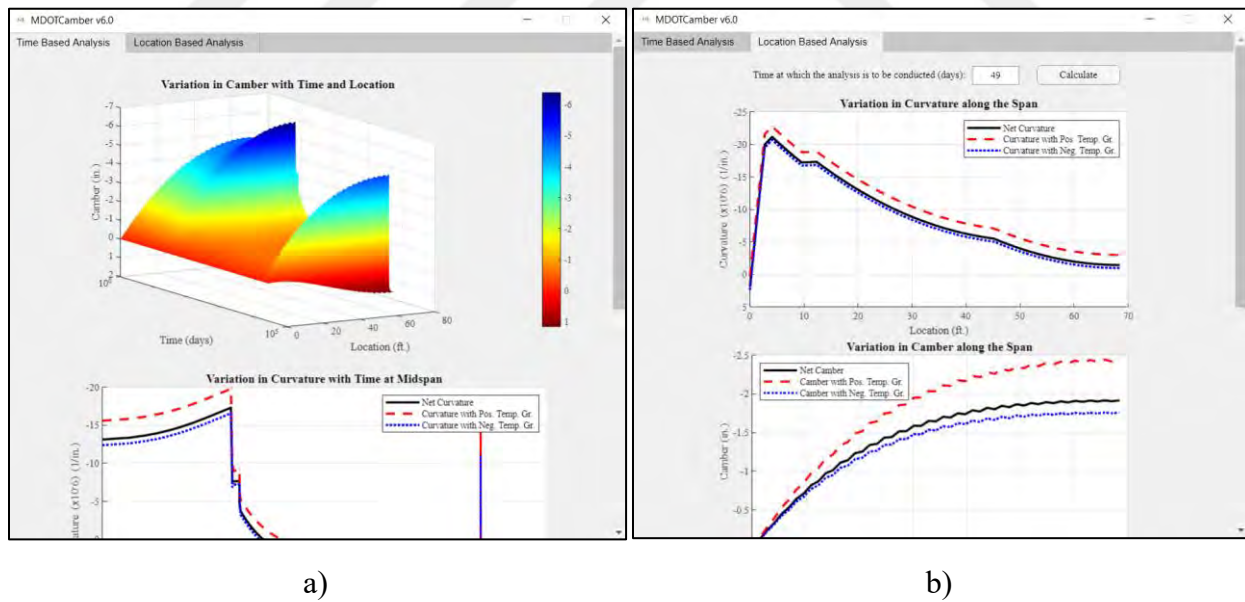
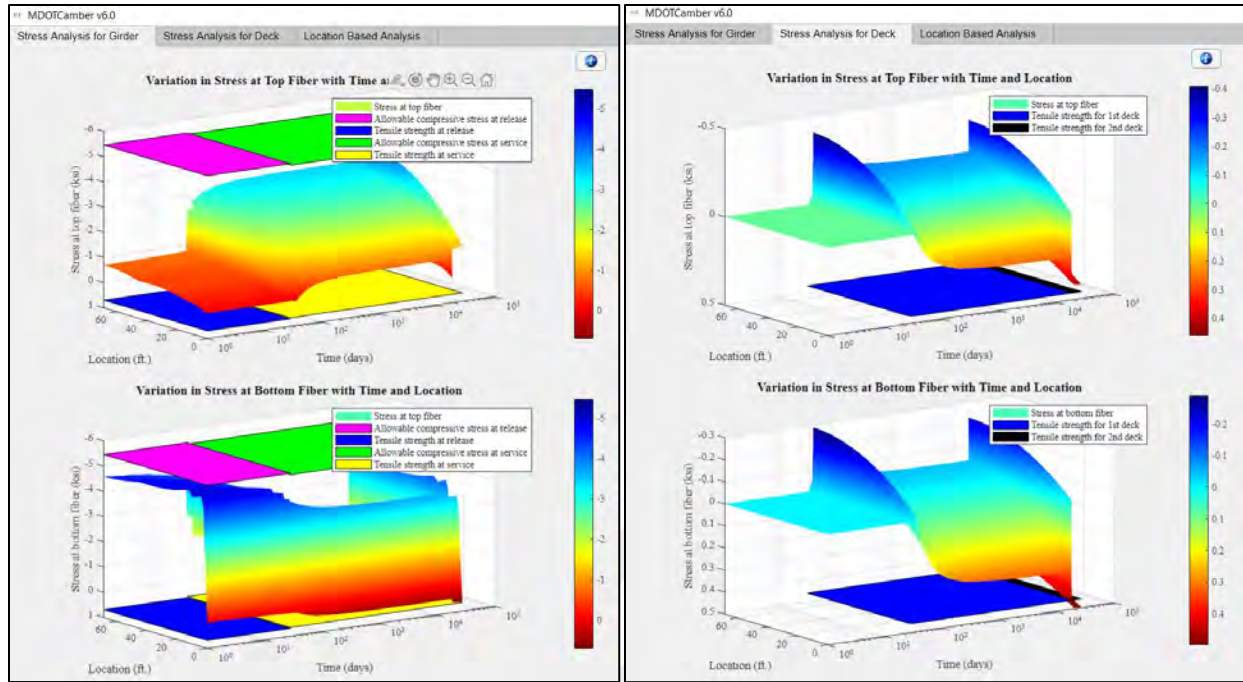
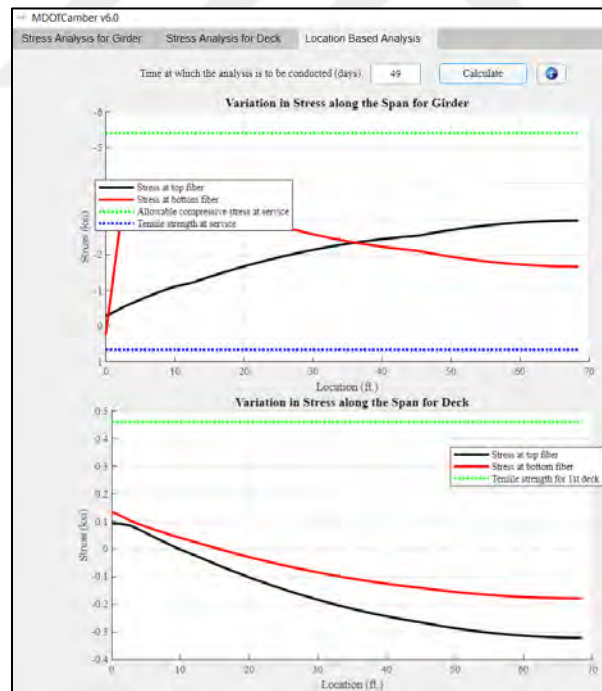


Figure G.17. Detailed analysis results for: a) Time-based analysis, and b) Location based analysis



a)

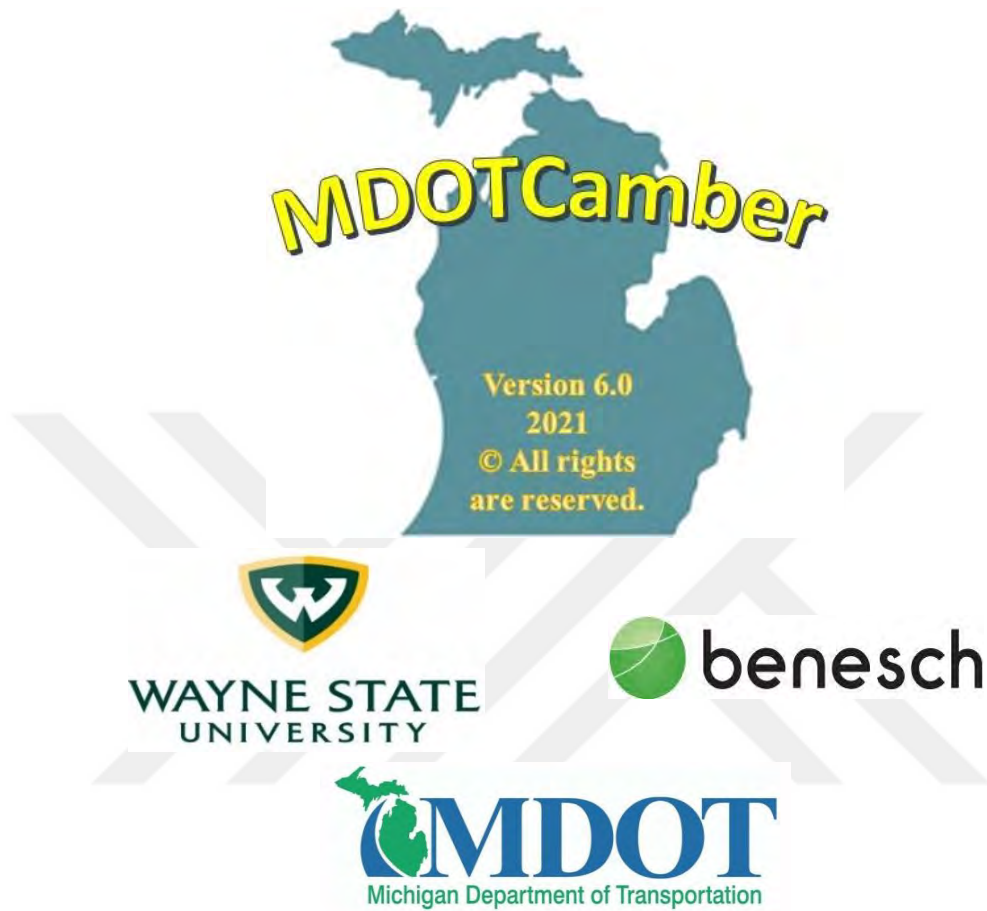
b)



c)

Figure G.18. Detailed analysis for stresses for: a) Girder, b) Decks and c) Girder and Decks along the half-span at given time

APPENDIX H: MDOTCamber v6.0 Installation Manual



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The link to access the executable file for the *MDOTCamber* program v6.0 can be requested from the author. When the link is clicked, it opens the page shown in Figure H.1. By clicking the “Download” button shown inside the yellow box below, the executable file can be downloaded.

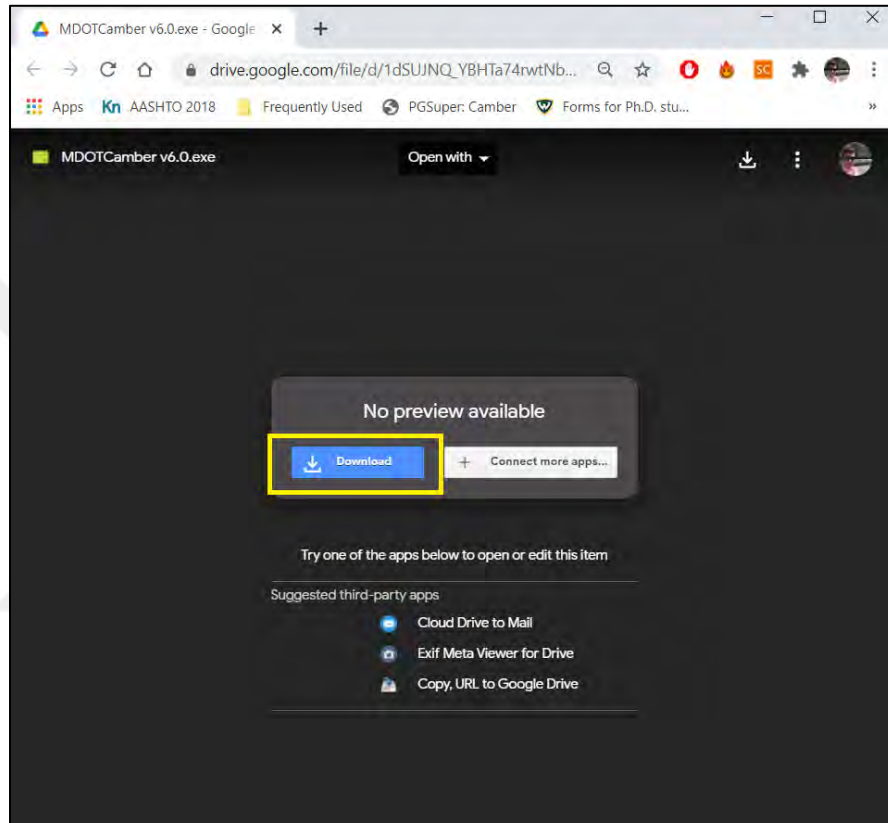


Figure H.1. Downloading *MDOTCamber* program

When downloading process ends, the computer may warn user as shown in Figure H.2. Then, user should click on the right arrow and click “Keep” option. The program itself is 12.0 MB. With this, the downloading process of *MDOTCamber* program is completed. The user should have the standalone executable file as shown in Figure H.3.

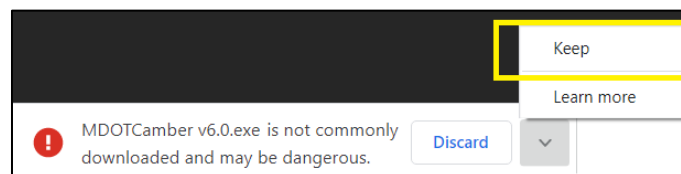


Figure H.2. Warning due to .exe file extension

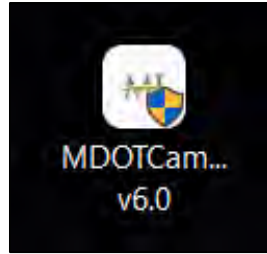


Figure H.3. Standalone executable file for the installation of *MDOTCamber* v6.0 program

To continue the installation process, user should run the executable file shown in Figure H.3. When tried to run, the computer may give another warning due to “.exe” file extension, as shown in Figure H.4. User should click on “More Info”, and then “Run anyway” buttons as illustrated in Figure H.4. If user has an antivirus program or another firewall protection in his/her computer, it may prevent the opening of *MDOTCamber* program. In this case, user should deactivate such virus protection programs temporarily.

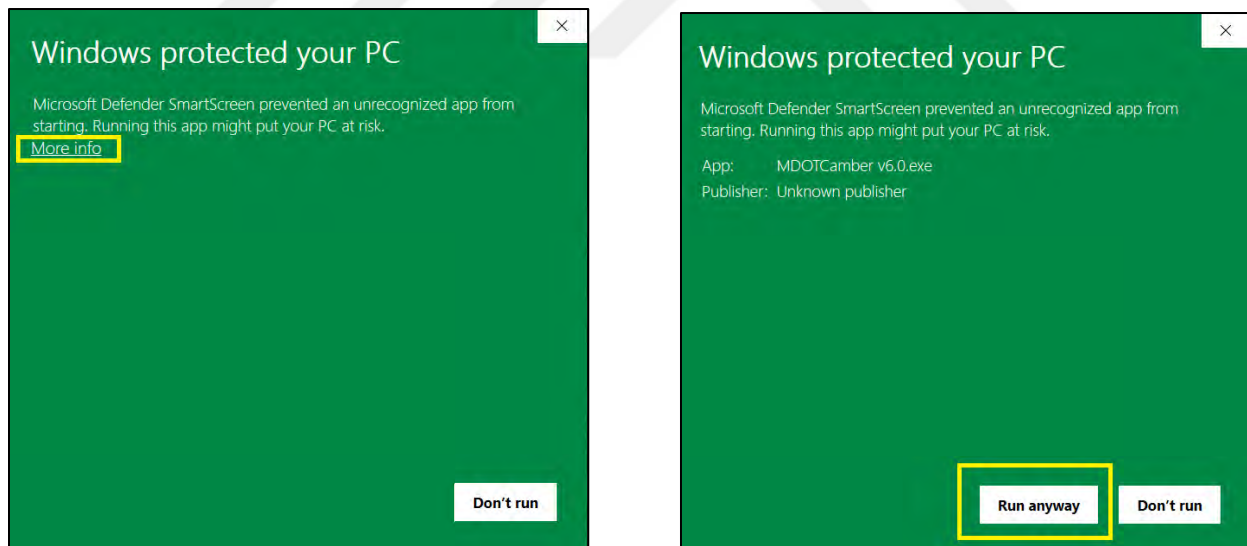


Figure H.4. Warning due to .exe file extension

After, a new window comes out as shown in Figure H.5. User should click on “Next” button, and this opens another window as shown in Figure H.6. To have a shortcut of *MDOTCamber* program in the desktop, user should select the option for this, and push “Next” button to proceed, as shown in Figure H.6.

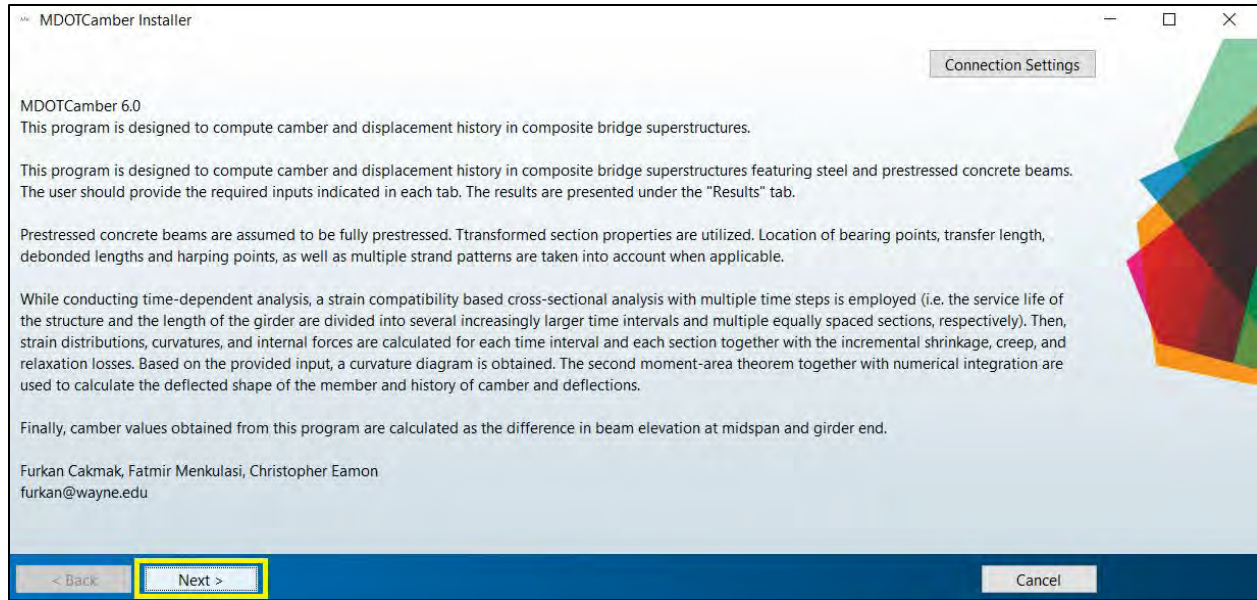


Figure H.5. *MDOTCamber* Installer

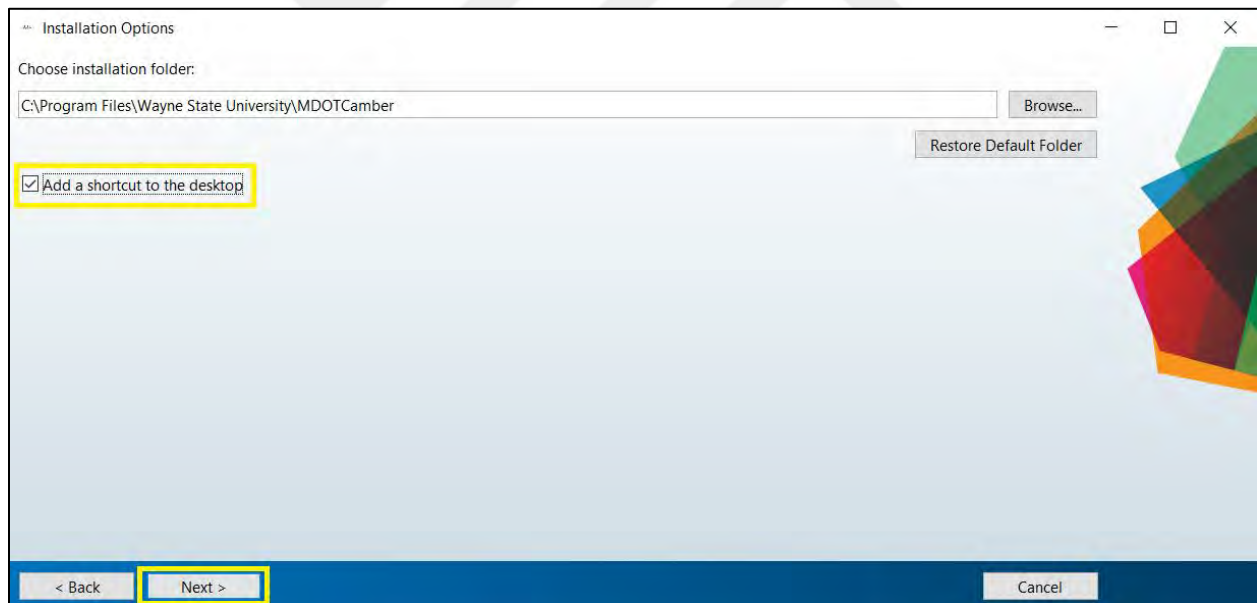


Figure H.6. Installation Options

The *MDOTCamber* program was written in Matlab R2020b. To be able to install the program, user should have Matlab R2020b runtime, which can be downloaded during the installation process of *MDOTCamber* program. User do not need to have Matlab license to be able to download the runtime. Program asks you to where to install the Matlab R2020b Runtime and suggests a default folder for this, as shown in Figure H.7. If intended, it can be downloaded prior

to installation of *MDOTCamber* program using the official website of the MATLAB. User should make sure that the correct version (version that is compatible with the current *MDOTCamber* program) of Matlab Runtime is downloaded. User should click on “Next” button to proceed. This leads to open a window shown in Figure H.8 about Matlab R2020b Runtime License Agreement. After reviewed, user should accept the license agreement, and click on “Next” button.

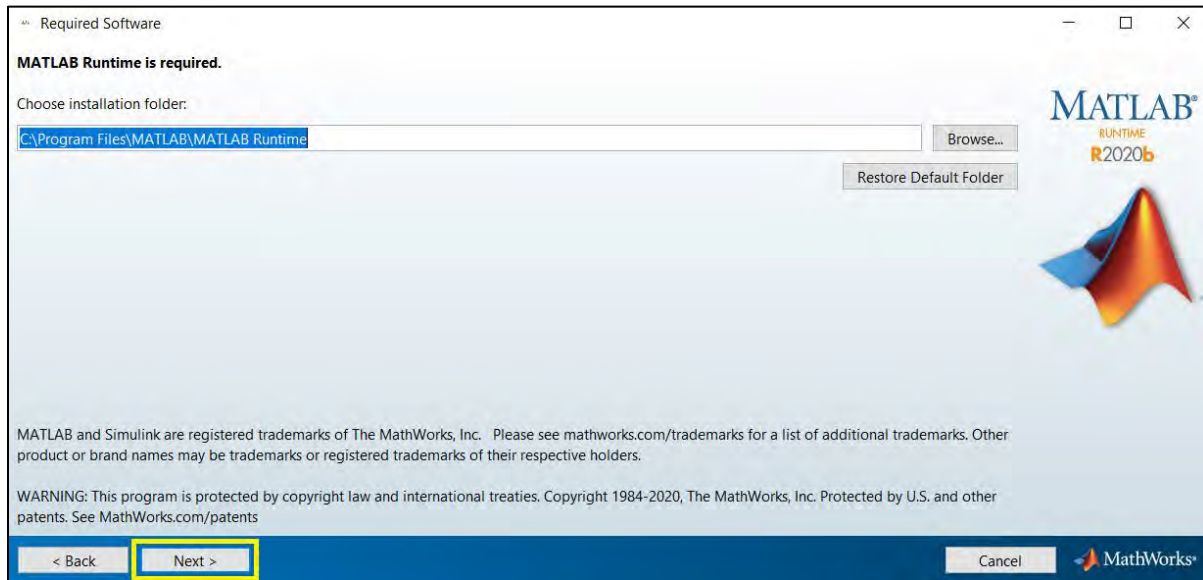


Figure H.7. Matlab R2020b Runtime

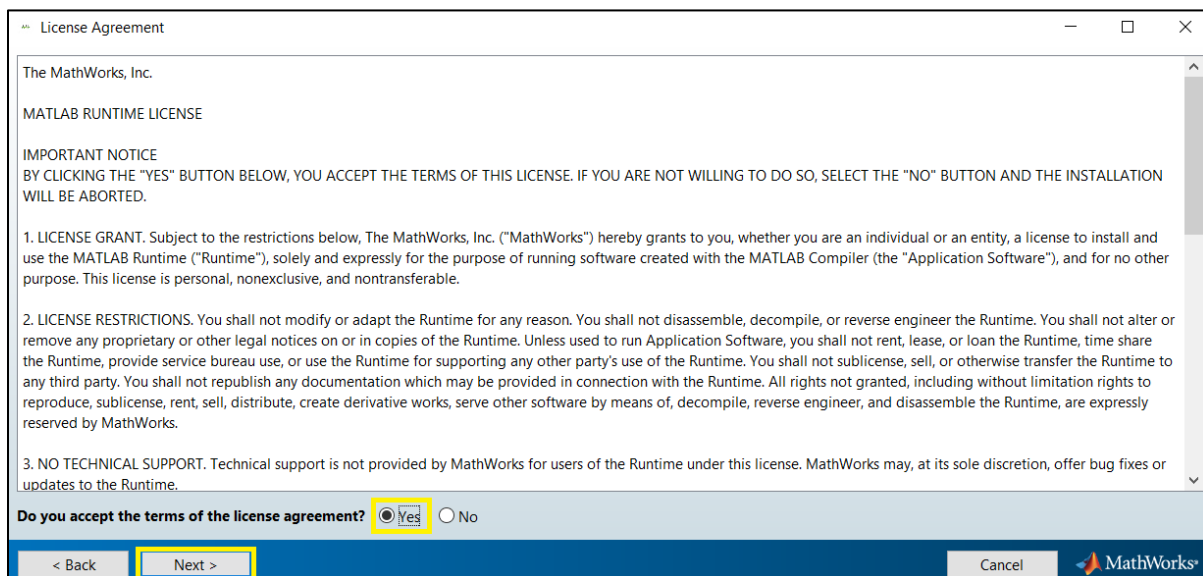


Figure H.8. Matlab R2020b Runtime License Agreement

A new window appears as shown in Figure H.9 for the final step before installing Matlab R2020b Runtime. As can be seen from Figure H.9, the runtime download size is 690 MB. This is only one time required process such that when a new version is released, user will not need to re-install the runtime unless it is not deleted or another version of Matlab is used. Depending on user's download speed, this process may take some time. By clicking "Install >" button, installing process may be initiated.

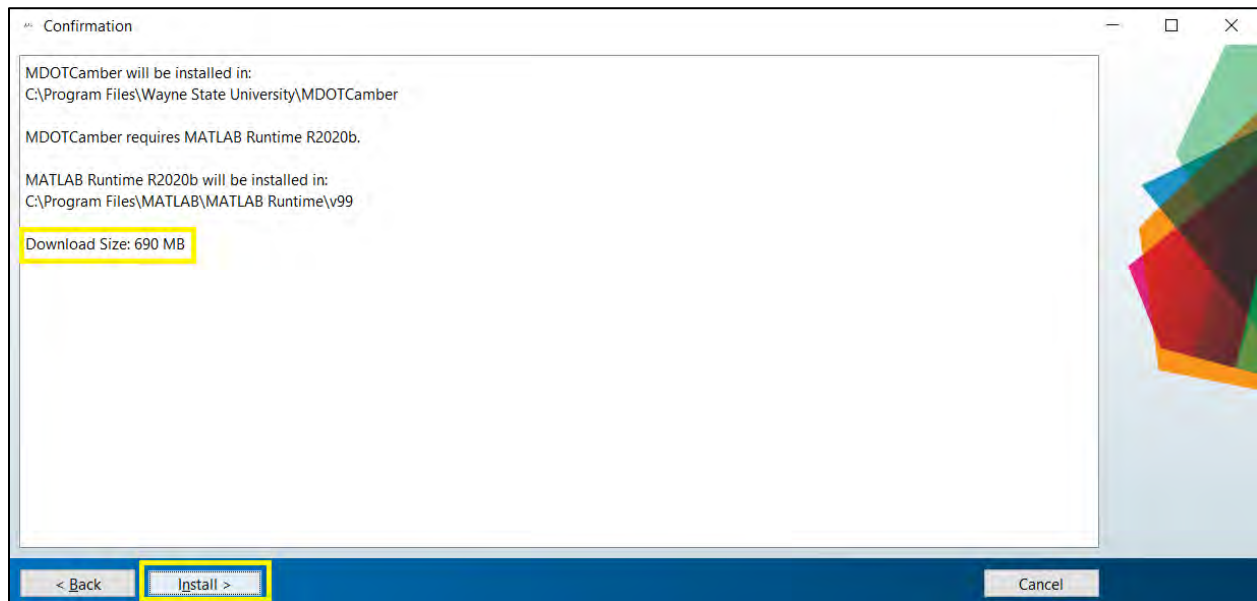


Figure H.9.Confirmation of Installation of Matlab R2020b Runtime

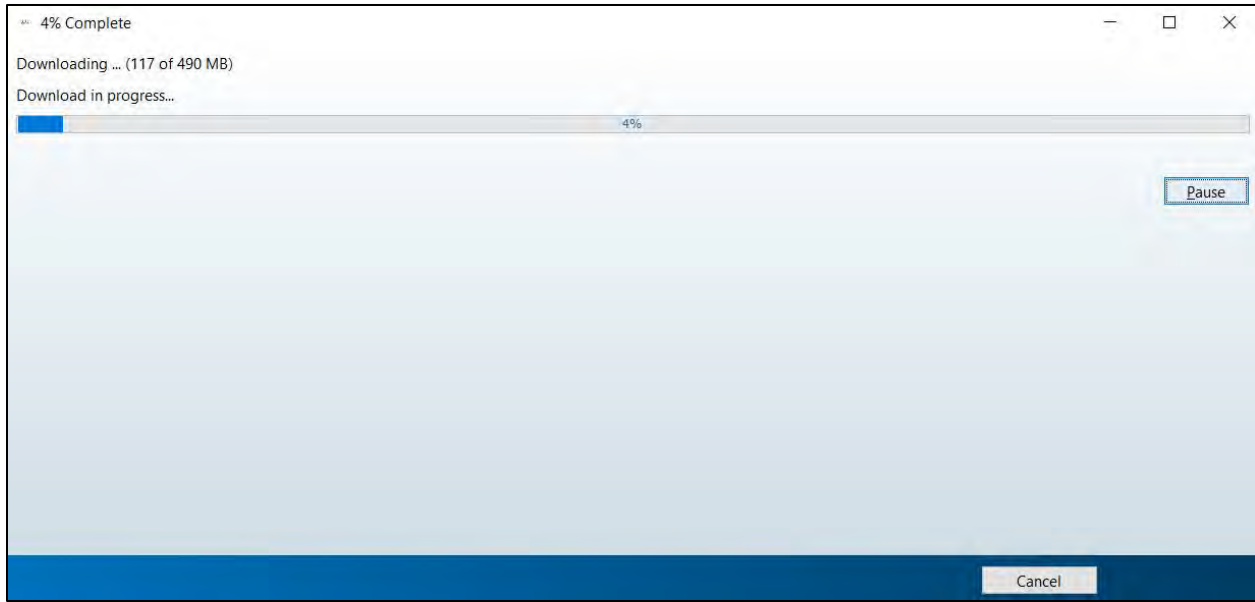


Figure H.10. Installation of Matlab R2020b Runtime and *MDOTCamber* program

After downloading process is finished, a window comes out showing that the installation process is completed as shown in Figure H.11.



Figure H.11. Installation complete

After clicking the “Finish” button, user should have the icon of *MDOTCamber* program as shown in Figure H.12. By double clicking on this, the *MDOTCamber* program can be opened. For

further assistance about how to use the *MDOTCamber* program, please consult user's manual which is provided in APPENDIX D.

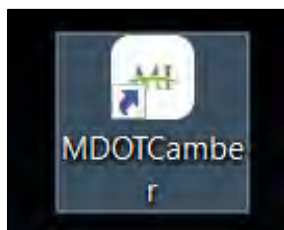


Figure H.12. Icon for *MDOTCamber* Program

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ABSTRACT**EVALUATION OF TIME-DEPENDENT CAMBER AND DEFLECTIONS IN BRIDGE GIRDERS**

by

FURKAN CAKMAK**May 2021****Advisor:** Dr. Fatmir Menkulasi**Major:** Civil Engineering**Degree:** Master of Science

A methodology to predict each stage of camber and displacements in composite prestressed concrete and steel bridge superstructures from inception to end of service life including a prediction of rebound in deck replacement projects is presented. The methodology invokes the use of multiple creep curves and allows the proper simulation of various construction activities each of which feature unique loading events. The proposed method captures the influence of several factors, such as creep and shrinkage of beam and deck concrete as well as prestresses losses induced because of these phenomena. Additionally, the prediction methodology offers the capability to quantify the influence of temperature gradients on beam camber and displacements at any point in time. The proposed method can be used to predict beam rebound during a deck replacement activity by considering the removal of the deck as well as the removal of the locked-in time dependent internal forces. The overall framework for the prediction methodology is based on principles of engineering mechanics, although components of the methodology are based on empirical models such as the estimation of the modulus of elasticity at prestress release and 28 days, its variation with time, prediction of creep and shrinkage properties, and relaxation of prestressing strands. The proposed methodology has been evaluated using measured pre-erection

and service camber data. An alternative simplified method suitable for preliminary design and based on time-dependent multipliers is presented. Pre-erection camber predictions based on the proposed methodology and the Time-Dependent Multiplier Method are more accurate than those based on the PCI Multiplier Method and the MDOT multiplier method. A computer program called *MDOTCamber* was developed to facilitate data input and summarize essential output.



AUTOBIOGRAPHICAL STATEMENT

Furkan Cakmak was born in 1996 in Istanbul, Turkey. He got his bachelor's degree from Middle East Technical University, METU, in Civil Engineering. In the last semester before he graduated, he went to Strathclyde University, located in Glasgow, United Kingdom, as an exchange student and gained valuable experiences about studying abroad. In the summer when he graduated from METU, he got accepted by Wayne State University as a Graduate Research Assistant. During his graduate studies, he studied the prediction of time-dependent camber and deflections of bridge girders, and developed a computer program called *MDOTCamber*. In this study, he proposed an advanced prediction methodology to predict the time-dependent deformation in composite/non-composite prestressed concrete girder and composite steel girders. He spent about 2 years on this study and, this thesis is the culmination of the effort he put in this research. For PhD studies, he plans to continue studying time-dependent behavior of prestressed and post-tensioned concrete.